A FIELD STUDY OF FACTORS CONTRIBUTING TO THE RUTTING AND CRACKING OF ASPHALT PAVEMENTS IN MONTANA

Prepared for the
State of Montana Department of Highways

February, 1990

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REPORT DOCUMENTATION PAGE

Abstract

This paper details a series of studies to determine the possible relationships among a variety of pavement variables and rutting or transverse cracking in the Montana Interstate System.

A pilot study to assess the number of samples and sampling pattern required for a statistically-significant conclusion is described.

For the main study, about 185 paving projects in the Interstate System were surveyed and sampled. A large variety of physical parameters including asphalt content, aggregate gradation and air voids content as well as results from high performance gel permeation chromatography (HP-GPC), traffic and climate factors, etc., were subjected to multiple regression analyses to find possible models for relationships among such factors and rutting or transverse cracking.

The data set was divided into several sets, pavements with no overlays, pavements with overlays, and pavements with open graded friction courses. Models for the relationships of measured factors with rutting and with cracking are described.

Keywords: Asphalt Concrete; Rutting; Cracking, transverse; pavement variables; physical properties; HP-GPC; statistical analysis; statistical modeling.

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DISCLAIMER STATEMENT

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Montana Department of Highways or of the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

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TABLE OF CONTENTS

EXECUTIVE SUMMARY

A total of 185 sites, corresponding to paving projects on the Montana Interstate Highway system, were surveyed and sampled for this research project. The project was intended to determine factors which are associated with rutting and transverse cracking in those pavements and relate them via statistical models.

It was necessary to separate the sites into three categories: non-overlayed pavements; overlayed pavements; pavements with open-graded friction courses. This arose because statistical models could not be constructed with the full data set, presumably because different factors are involved.

A variety of laboratory and historical data as well as performance observations were assembled for each sample site. This data was then treated by statistical regression analysis with total rut depth as the dependent variable. A model for rutting in non-overlayed sections of Montana Interstate highways that included percent fractured faces, percent voids, percent aggregate retained on the 10M screen, asphalt content, percent aggregate passing the 200M screen, penetration at 77°F, and both absolute and kinematic viscosities was developed. Some of the predictor variables were modified by reference to age or to commercial traffic factor. Climate region, specifically average August temperature, was also found to be significant. R^2 value for the model was 0.5880, which is considered to be reasonable for a data set of this type.

The model was used to predict the extent of rutting for each site. Predicted values were then compared with measured values. The resulting residuals showed that in most cases, predicted rut depths were within ± 4 mm of actual values and all but 3 were within ± 6 mm of measured depths.

By inserting different values for the dependent variables into the model equation, expected rut depths may be calculated. From this exercize it appears that rutting may be eliminated or, at least, reduced by judicious attention to details in the predictor variables. Difficulties with this process will be presented by the interdependence of certain variables such as penetration and viscosity and some compromises will, no doubt, be required.

Similar processes were undertaken to model rutting in pavements with overlays and transverse cracking in non-overlay pavements. The results from these efforts were less satisfactory, however.

The statistical modeling of rutting in overlay pavements is hampered to some degree by the narrow range of ages and rut depths as well as of other variables in the subject pavements and

by the lack of data on open graded friction courses associated with these overlays.

Nevertheless the models obtained emphasize the importance of aggregate gradation in the overlays. They also indicate a contribution from a lower temperature parameter, the penetration at 40°F. Moreover, the models show that certain characteristics of the original pavement may influence rutting in the overlay. It is unfortunate that data on the rut depths in the original pavements at the time of overlay is not available.

A model for transverse cracking contains some of the same parameters found in the rutting models. Nevertheless, firm conclusions about the relationships of various factors to cracking remain difficult to draw, perhaps because of the variety of variables affecting the asphalt itself and the uncontrolled nature of the data set. The Test Sections seem to provide better models for transverse cracking.

It should be noted that there may be other factors which influence rutting and cracking but are not found in the models. This could occur if there is a limited variability for the factor in the data set and it is therefore ignored by the analysis. Such limited variability could arise if, for example, the parameter has been adequately controlled by MDOH in the past. Too, the factor may not have been measured.

Nevertheless, the statistical model for rutting, particularly for non-overlay pavements that has been generated is reasonable and potentially quite useful.

The results of the studies do not present the ultimate solution to the rutting and cracking problems in Montana but it is reasonable to state that by adjusting factors pinpointed by the studies and keeping high quality construction and design standards with regard to other factors, a reduction in rutting and perhaps in cracking can be achieved.

INTRODUCTION

The problem of rutting in flexible pavements continues to be a focus of attention in Montana and nationwide. More than most other forms of pavement deterioration, rutting constitutes a safety hazard. Repair is difficult and expensive and that is aggravated by the fact that rutting often develops rapidly, requiring maintenance within a few years after construction.

Potential causes for rutting are numerous and several studies have focused on various aspects of the problem (1-4). Intuitively, one suspects that there are multiple causes working alone or in tandem. Obviously, no distortion of pavement would occur were it not for the stresses imposed by heavy traffic. Since traffic is likely only to grow heavier, one might be discouraged were it not for the fact that some pavement sections resist deformation under the same stresses to which neighboring sections succumb.

At the same time, transverse cracking continues to be a problem in the state. Previous studies of this mode of distress have concentrated on the asphalt cement itself and have not included all geographical and climatological regions in the state.

With this in mind, a field study has been undertaken to identify factors common to rutted pavements but not found in resistant pavements on the Interstate Highways system in the State of Montana. A secondary focus was to identify factors

associated with transverse cracking. The results of these studies will be discussed in this paper.

Background

In recent years, rutting has come to be a recognized problem on Montana roads. In testimony to the regional significance of the problem, WAASHTO (Western Association of American State Highway and Transportation Officials) appointed a Rutting Task Force in 1983. This group, after considerable deliberation, recommended a series of steps that it felt would be helpful in minimizing the underlying causes of rutting (5). Montana Department of Highways (MDOH) responded to this study by adapting the recommendations to its own situation and instituting a set of anti-rutting specifications (6) in 1985. Early evaluations of the performance of pavements constructed under these specifications have been encouraging, but the long-term efficacy of the new specifications has yet to be determined.

At the same time, the importance of determining the actual causes of rutting in the state was recognized. The current study was proposed for this purpose. Several field studies of transverse cracking in Montana and selected other states have been done by this research group. As stated earlier, these have been restricted to the asphalt cement. The scope of the rutting study offered an ideal opportunity to extend the factors considered in the causes of transverse cracking.

A field study is very complicated because of the large number of variables which exist. This can be understood as an

advantage, however, in that a field study considers "real world" situations, and effectively turns the highway system into a large laboratory in which performance is an accomplished fact. Factors contributing to the performance are there to be found, if the proper questions can be asked.

In this paper, the design of a field study to determine the causes of rutting and, secondarily, of transverse cracking on the Interstate Highway System in Montana will be discussed. The results of the studies will then be presented.

I. STUDY DESIGN

In the design of a study to determine the causes of rutting in Montana, several factors were considered. The study should:

1. take advantage of the varied responses of Montana pavements to rutting pressures;

2. be broad enough to uncover what are perceived to be different causes of rutting in various parts of the state;

3. evaluate pertinent common design and physical parameters routinely used by MDOH as well as molecular size distribution of the asphalt cements by HP-GPC;

4. be statistically viable.

In response to requirements 1 and 2, the Interstate System was selected as the population from which to draw samples. This would permit sampling of pavements of different ages from different aggregate sources, etc., but that were constructed under similar basic design philosophies.

Parameters chosen for evaluation were:

Bulk specific gravity

Rice specific gravity

% voids

Marshall stability and flow

% asphalt

Aggregate gradation

% aggregate with fractured face

Penetration at 40°, 77°, and 90°F

Viscosity at 140° and 275°F

Ductility at 77°F

HP-GPC analysis

These tests focus on characteristics thought to contribute to pavement instability.

It was necessary to take into account the variation in these parameters resulting from inconsistencies in the pavement and/or reproducibility of the test procedures. Although some statistical studies have been done on such variability, none was sufficient for the purposes at hand. Therefore, a two-part project was initiated. First, a limited pilot study was used to evaluate these variables and provide information needed for the design of the second and main phase of the rutting study.

A. Sampling design, pilot study

In consultation with a statistician (Dr. Robert Boik, Department of Mathematics, Montana State University) a practical sampling pattern that would provide the needed data was designed.

One paving project was selected in each of three categories: ruts > 0.75 inch; ruts 0.50 to 0.75 inch; ruts insignificant. This was done to maximize the possibility in so small a sample that differences, if present, could be observed.

The number of sampling sites within each project depended on the overall length of the project, i.e., a 4-mile long project contained two sampling sites whereas one of 11 miles had six sites. Sites were defined as mile posts and were randomly selected.

Core samples were collected from the outside wheelpath of the driving lane and from the adjacent shoulder at each site. At one site in each project, a total of 18 cores was obtained; at other sites, six cores were collected. This pattern is diagrammed in Figure 1.

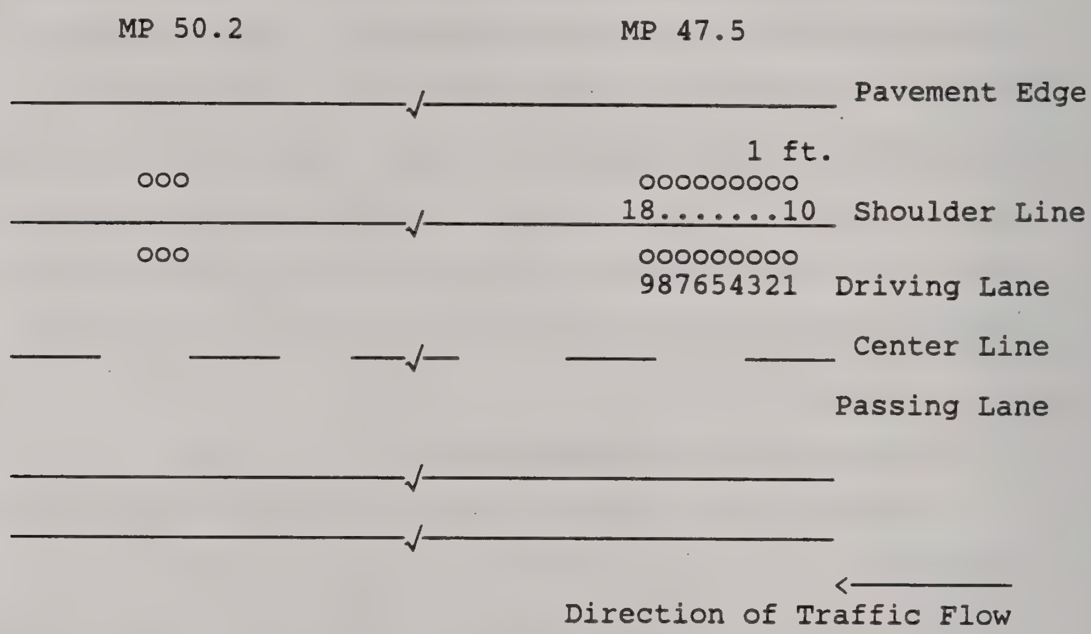


Figure 1. Sampling pattern for pilot study.

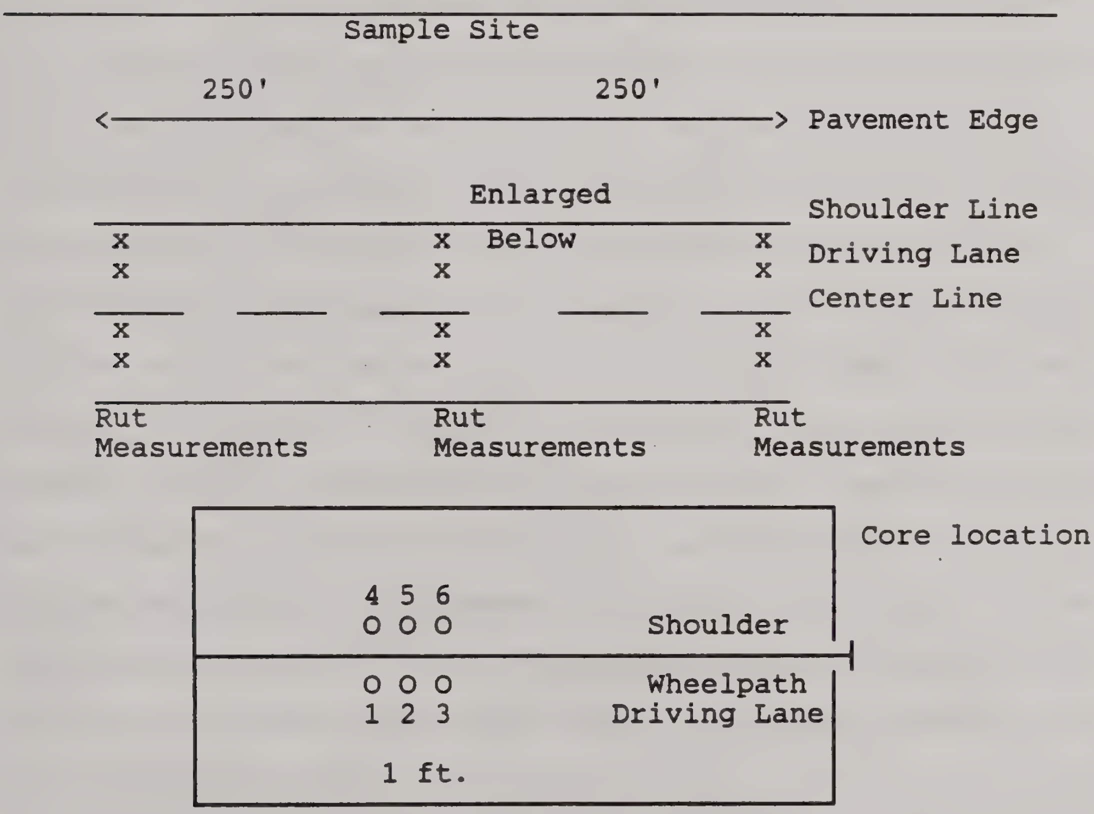
The results of the pilot study and their significance for the main rutting study will be discussed in detail in section II.

B. Sampling Design, Main Study

Once the pilot study data was in hand, the remaining work could be planned. It was decided that one sampling site in each

of 180 paving projects on the Montana Interstate Highway System would be sufficient for a statistically sound determination of the factors contributing to rutting.

Sampling sites were selected at random by computer within each paving project. A team of researchers visited each site to observe the overall condition of the pavement and to measure rut depths and count transverse cracks. This team marked each site with paint for the convenience of coring crews. Sampling pattern and location for rut measurements are detailed in Figure 2.



<u>Figure 2. Sampling and Observation Pattern, Main Study</u>

Historical data was sought for all sites, particularly mix design, construction and pavement design information. Such material was not always available, however.

Laboratory tests were conducted according to standard procedures used in MDOH laboratories except for HP-GPC analyses procedure which will be included in Appendix B.

All data was subjected to statistical analyses in the effort to find correlations between the rutting performance observed and the parameters investigated. The nature and results of this manipulation will be detailed in section III.

II. THE PILOT STUDY

As mentioned earlier, three paving projects with widely differing rutting performance were selected for the pilot study. It must be emphasized that the performance criteria were set in order to maximize the possibility of being able to detect differences in parameters tested, if the differences exist. In so small a sample, no correlations may be made between the parameters and performance.

A. Data

A great deal of data was generated in the course of this pilot study. The raw data will not be presented here, but summaries of each test as well as historical data will be found in Appendix A. There are some observations that should be brought out. For example, there appears to be a wide variation in the results of the Marshall Stability test done on pavement cores. In Project 3, driving lane, these values range from 982 pounds to 2526, a difference of 1544 pounds. The smallest range is 267 pounds. Inherent in this variability are both inconsistencies in the pavement and testing variability. The test was designed for use on laboratory prepared specimens and its use on roadway cores which vary in thickness requires the use of correction factors.

Penetration at 77°F, the industry standard, varies in these pavements by as much as 53 units, more than 100% of the lower

value. In spite of this however, the means permit differentiation among these three projects with a high degree of confidence (P value = 0.006).

Asphalt content was determined by two procedures, by difference (recovery of the aggregate and then subtracting the weight of aggregate recovered from the original weight of the sample - the standard method) and by direct means (recovery of both asphalt and aggregate). The percentages obtained by the direct method are, in most cases, significantly different from those obtained by difference, and are always lower. Both methods show that there is a narrow range of asphalt contents in Project 1 (F0.1%) but a much wider range than would be desired in Project 2 (2.2%).

Voids content is remarkably consistent in Projects 1 and 2 but widely variable in Project 3. That, combined with the fact that differences between projects are fairly small, means that although the P values relating to the significance of differences between the driving lane and shoulder are on the order of 0.02, the difference between projects has a less encouraging P value of 0.5.

Table 1 contains a summary of P values for the tests performed.

Table 1. Pilot Study - P values

Variable	Lane[1]	Proj. x Lane[2]	Proj.[3]
Marshall Stability	0.14	0.44	0.29
Marshall Flow	0.16	0.28	0.06
% Asphalt	0.63	0.28	0.01
% Asphalt by diff.	0.66	0.91	0.002
Pen 40°F	0.06	0.69	0.0008
Pen 77°	0.05	0.51	0.005
Pen 90°	0.01	0.05	0.05
Vis 140°F	0.35	0.94	0.08
Vis 275°	0.02	0.29	0.07
Ductility 60°F	0.003	0.003	0.21
% LMS	0.205	0.291	0.03
% MMS	0.215	0.209	0.259
% SMS	0.192	0.317	0.008
Bulk sp. gr.	0.03	0.73	0.006
Rice sp. gr.	0.82	0.94	0.40
% Voids	0.02	0.49	0.39

P-values relate to the level of confidence with which we can believe that the differences observed do not result from chance:

(1) for differences between driving lane and shoulder;

(2) for differences between projects based on difference between driving lane and shoulder;

(3) for overall differences between projects.

The smaller the value of P, the more likely it is that the differences observed do not result from chance. Ideally, P should be less than 0.05.

B. Significance for the main rutting study

The purpose of the pilot study was, of course, to permit the design of a major study to determine the causes of rutting in the State of Montana. Some of the statistical ramifications of the pilot study data toward that end are recorded in Table 2. Included are values of the variance (σ^2) and 2δ for each variable.

About 70% of the observations for a given variable in a single cell (see Figure 3) should be within the interval: mean ± σ (standard deviation); 95% should then be within the range: mean ± 2 σ. For example for Marshall Stability measurements, 75% of the samples in a given cell should be within the range: mean ± 935, and 95% in the interval mean ± 1870.

The values for 2δ indicate the size of differences between samples that may be considered to be significant based on the pilot samples. For example, with a sample size of 117 (Table 2.) differences between Marshall Stability values of 780 (2δ), or ± 390 should be significant. To decrease this range to ± 300, a total sample size of about 200 would be required. Similar analyses can be made for each of the variables measured. Voids content has, historically, been considered to be a major factor in the amount of rutting observed in a pavement. Therefore, it is considered essential that a large enough sample size be taken to ensure that significant differences in voids content can be observed. For a sample size of 117, differences of 2% (± 1%) should be observable; for the larger sample size, that range should be reduced to 1.5% (± 0.75%).

Table 2. Statistical Data from Pilot Study

	$\hat{\sigma}^2$	2δ for n=117	2δ for n=198
Marshall Stability	87.4	780	590
Marshall Flow	5.9	2.0	1.5
% Asphalt, direct	0.14	0.31	0.24
% Asphalt, by diff.	1.7	1.1	0.8
Pen @ 40°F	4.4	17.5	13.2
Pen @ 77°F	36.2	50.0	38.0
Pen @ 90°F	4.9	18.0	14.0
Vis @ 140°F	106	860	650
Vis @ 275°F	0.62	66.0	50.0
Duc @ 60°F	7.6	23.0	17.0
% LMS	0.96	0.8	0.62
% MMS	0.12	0.3	0.22
% SMS	0.41	0.53	0.40
Bulk sp. gr.	2.7×10^{-4}	0.014	0.010
Rice sp. gr.	1.8×10^{-4}	0.011	0.0084
% Voids	5.7	2.0	1.5

Figure 3. Sampling Pattern Proposed

Age	Ruts,mm None	0.5-0.75"	>0.75"
0-5 Yrs.	22	22	22
6-10	22	22	22
>10	22	22	22
No. of Sites, n	66	66	66

III. THE MAIN STUDY

The major focus of this research project has been, of course, to determine the causes of rutting and, secondarily, of transverse cracking by means of a field study, that is, an investigation of pavements in service. That field study is the subject of this section of the report. The discussion will be divided into several portions:

A. Data Collection, including site selection, performance observations, historical search and laboratory work.

B. Characteristics of the Data, including a discussion of each of the parameters considered.

C. Statistical Model for Non-overlay Pavements, in which various aspects of the analysis of this group of pavements will be found.

D. Statistical Models for Rutting in Overlays, detailing the work on this class of pavements.

E. Statistical Model for Transverse Cracking, which uses the same data base, will conclude the report.

A. Data Collection

Site selection. For the main portion of the rutting study it was decided, as mentioned earlier, to use only the Montana Interstate Highway System. The main considerations in this selection were the availability of a large sample size and, even more important, the availability of paving projects that were constructed under very similar design philosophies. For example, it was expected

that the requirements for subgrade and base course construction would be uniform.

A population of about 198 sampling sites were sought. Using the "Montana 1983 Federal Aid Road Log," it was determined that about 185 individual asphalt concrete paving projects (i.e. contracts) existed in the Interstate system. These varied in age and, presumably, in condition. Therefore, it was decided to select a sampling site in each of these paving projects.

Sampling sites within each project were chosen by a random number generator computer program. During the Spring of 1986, a team of three researchers visited every site. Location of each site was determined by use of highway mile markers and the automobile odometer to an accuracy of about 0.1 mile. In the few instances in which the randomly-generated site occurred on a bridge, the site was changed.

The location of the first core at each site was marked with white enamel spray paint as an "X" in the outside wheelpath at the mile marker. (See page 6 for a layout of core positions). The mile-post number was written on the shoulder to facilitate labeling of cores by the coring crew. As noted earlier, six cores were collected at each site, three in the outside wheelpath of the driving lane and three in the adjacent shoulder. All coring was done by MDOH crews using water-cooled core drills. Cores were tagged and shipped to MSU laboratories.

Performance Evaluations

Rut depths were measured at the coring site and at two additional locations about 250 feet on either side of the coring site. Extent of deformation was noted in the inside and outside wheelpaths of both driving and passing lanes. Measurements, in millimeters, were taken using a stringline stretched from inside shoulder line to outside shoulder line, except for a few instances when banking (superelevation) of the roadway necessitated basing the stringline from centerline to shoulder. The number of transverse cracks was also recorded and will be discussed later. Other characteristics, less quantifiable, were also noted, including flushing, roadway grade and curvature, type of terrain, proximity to interchanges, etc.

Historical Information

Several sources of information housed at MDOH in Helena were searched for data on the sampling sites. These sources included "Straight Line Diagrams," "True Mileage" records, "As Built" records, construction files and "Special Pavement Condition Inventories." From these records such important information as year of construction, designed pavement depth and extent of rutting before overlay, as well as a variety of other design and construction information was obtained.

Unfortunately, several factors limit the usefulness of some of this data. The form in which some records are kept has changed and some records are incomplete, probably reflecting, at

least in part, changes in emphasis at MDOH with time. Sometimes records of one type disagree with those of another and it is difficult to pinpoint the construction project associated with a particular sampling site. In a few instances, the depth of pavement found in the cores contradicted the amount accounted for in the records. However, these problems have not impeded progress toward the main goal of the project.

Laboratory Procedures

Cores were cut from the roadway, tagged and shipped directly to the MSU laboratories. Upon arrival, they were logged in, total core thickness was measured, obvious divisions between paving operations (lifts) were marked and measured. These were then compared with information on paving history and necessary adjustments were made. The cores were then separated into individual lifts by means of a water-cooled saw. Open graded friction courses were removed and discarded, as was the uppermost 0.25-0.5 inch of all exposed lifts.

Cores #1 and #4 at each site (driving lane and adjacent shoulder, respectively, see Figure 2, page 9) were used for bulk specific gravity measurements and for HP-GPC analyses. Cores #2 and #5 were used for Marshall Stability and Flow tests as well as for Rice Specific Gravity data. Cores #3 and #6 were reserved for extraction. From this material, data were obtained for the following tests: asphalt content, penetration, viscosity, ductility, sieve analysis and count of fractured faces.

Most of the procedures used were those accepted and used by MDOH and AASHTO. A few modifications were necessitated by the conditions of this experiment. For example, fractured face counts are normally done on "pit-run" type aggregate samples. Counting fractured faces in aggregates recovered from core samples is complicated by the presence of cut faces. To simplify the procedure, cut faces were colored while the core was intact with a dye which would withstand water immersion and trichloroethylene extraction.

A complete list of the procedures used, and notes regarding any modifications, as well as a complete procedure for HP-GPC analysis are included in Appendix B.

B. Characteristics of Data

Of the 185 sample sites making up the original sample set, 67 consisted of original pavement only, that is, had never received an overlay.

Several sites were located so close to the end of a project that it was not possible to determine the actual project under which they were constructed and the age difference in the adjacent projects was significant. Therefore, these sites were eliminated. Additional sites were situated in recycling projects where original pavement and recycled material were not sufficiently delineated for confident use of data; these were also eliminated. Total number of sites eliminated for any of these reasons was 7.

Open graded friction courses (OGFC) were present at 80 of the remaining sites. There has been a question of the association of stripping in underlying pavement with use of OGFC. It is of interest that cores from seven sites were either seriously eroded by the coring process or had portions totally disintegrated (i.e., a lift in which stripping was so severe that coring crews recovered only loose aggregate with no evidence of asphalt cement remaining). Of these, four were under OGFC. It is noteworthy that rutting in these pavements in which the OGFC's were 1-3 years old was negligible.

For the data set as a whole, rut depths ranged from 0 to 31 mm (0-1.2 in). Ages of the exposed lift ranged from 1 to 25 years. This data is plotted in Figure 4 in which those sections which have an overlay are coded "O", those consisting only of the original pavement (non-overlay) are marked "+".

A third symbol, "X", on this plot has been used to designate the approximate rut depths of some original pavements now covered with overlays. This rut depth data is taken from the Pavement Condition Surveys for 1983 or 1984, as appropriate. In those surveys, rutting was reported in three ranges: less than 0.5 inches, 0.5 to 0.75 inches and more than 0.75 inches. For purposes of this work, these three ranges have been converted as follows:

$$<0.5 \text{ in.} \equiv 6.5 \text{ mm}$$

$$0.5\text{-}0.75 \equiv 19 \text{ mm}$$

$$>0.75 \equiv 25 \text{ mm}$$

Because of the inexact nature of this data, these original pavements could not be used in generation of rutting models.

In Figure 4, dashed lines have been drawn which segment the sites into nine possible groups by age range and rut depth range, corresponding to the cells desired for the statistical analysis (see Figure 3, p. 16). It can be seen that some of the cells are empty, or nearly so, particularly those with more than 19 cm of rutting which are less than 10 years old. Although this fact may be less than encouraging for the statistical analysis, it should be somewhat reassuring to MDOH. That is, although there is more rutting than desirable, Montana apparently does not experience the explosive rutting rates, for example one inch or more in the first year, that have been seen elsewhere. Also, there are a significant number of pavements more than 10 years old that have less than 12.5 cm (0.5 inch) of rutting.

The location of "X" symbols in the upper right cell indicates that the dirth of sample sites in that cell, > 19 cm rutting and > 10 years, may be a result of the overlay program which figuratively moves these sites into the lower left corner of the plot. However, there are some sites which were overlayed but apparently were not seriously rutted ("X" in lower right cell). Keep in mind that the Pavement Condition Survey was begun in only 1983, so rutting data for pavements overlayed before that date is lost.

Initial efforts to model the pavements included the entire data set of uppermost lifts versus measured rutting. It soon

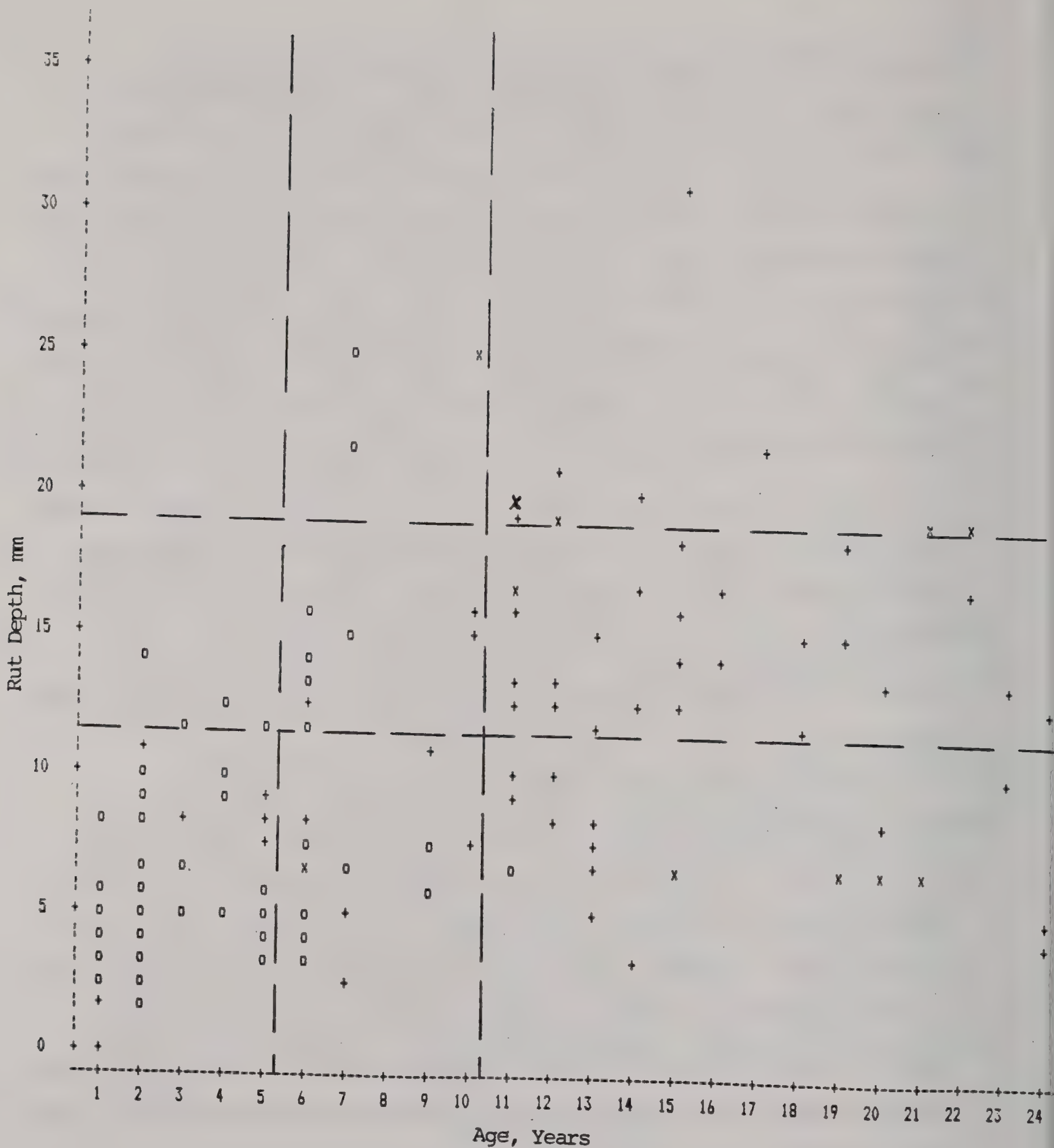


Figure 4. Plot of Rut Depth vs. Age, Whole Data Set

became apparent that a stable model could not be achieved. "Younger" roads appeared to rut at a different rate than "older" pavements. However, differences in age were not the cause of this problem so much as whether or not the pavement had an overlay. It was then decided to separate the sites into two categories: overlays and non-overlays (original pavements, never overlayed). It seems that models for the two categories are not the same, possibly because rutting in overlays reflects in some way the nature of the original pavement. Because of the potential complexity of this problem, it was elected to concentrate first on the analysis of original, non-overlayed pavements, and to return later to the problem of overlays and, finally, to cracking.

Abbreviations and Conventions

Before beginning the discussion of individual parameters, a list of those parameters, their definitions and abbreviations follows.

Design Marshall Stability - the value of Marshall Stability obtained in the pavement design process.

Construction Marshall Stability - the average or estimated average of Marshall Stability values obtained in the field from laboratory-compacted grab samples of hot mix.

Core Marshall Stability - the value of Marshall Stability obtained for individual lifts of cores taken from the roadway in 1986.

Design Asphalt Content - amount of asphalt cement required by the pavement design.

Construction Asphalt Content - average or estimated average of asphalt content in pavement at construction.

Core Asphalt Content - the percentage of asphalt cement recovered from roadway cores in 1986.

Commercial traffic factor - the number of commercial vehicles passing a site in 1985 multiplied by the age of the site, used as an estimate of the relative commercial traffic loads on the various section.

Design voids - the percentage of air voids prescribed by the pavement design.

Construction voids - air voids percentage obtained from field testing of grab samples of plant mix at the paving site. This value is an average or estimated average for the project.

Core voids - air voids percentage derived for cores taken at sampling site in 1986.

Aggregate passing x screen - the percentage of aggregate passing screen x in the sieve analysis.

Aggregate retained x screen - aggregate percentage retained on screen x in sieve analysis, but passing the next larger screen size.

Penetration at 40°F, 77°F or 90° - Pen 40, etc. - of recovered asphalt.

Absolute viscosity at 140°F - A vis - of recovered asphalt.

Kinematic viscosity at 275°F - K vis - of recovered asphalt.

Fractured faces - percentages of aggregate retained on 4 Mesh screen or larger having at least one angular fractured face.

Age factor - the age of the pavement at sampling site.

Viscosity temperature susceptibility - VTS - value for the recovered asphalt cement calculated by the formula:

$$VTS = \frac{\log\log(100\,N_1) - \log\log(100\,N_2)}{\log T_2 - \log T_1}$$

where N_1 = viscosity at T_1, poises

N_2 = viscosity at T_2, poises

T = temperature

Penetration Viscosity Number - PVN - temperature susceptibility calculated by the formula

$$PVN = \left(\frac{4.258 - 0.7967 \log P - \log X}{0.7951 - 0.1858 \log P}\right)(-1.5)$$

where P = penetration at 77°F, dmm

X = viscosity at 275°F, centistokes

Rutting depth - the average of depth of ruts in mm, in the more severely rutted wheelpath of the driving lane measured at the sampling site and at 250 feet on both sides of the sampling site.

Climate Regions - It is felt that climate, particularly summer temperatures, may have an influence on rutting. Therefore, an effort was made to assess this aspect of climate by

surveying the average air temperatures each month from 1951 to 1980 at U.S. Weather Bureau Stations along the routes of the Interstates. On average, August is the warmest month in this state. On the basis of this data, the area involved in the sampling was divided into four regions as follows:

Region	Average air temp., Aug. 1951-1980.
1	55.0-60.0°F
2	60.1-65.0
3	65.1-70.0
4	70.1-75.0

A map of the Climate Regions is on page 45.

Simple Correlations with Rut Depth

Simple correlations were made between each of the variables and rut depth. As expected, correlations were found with age of pavement ($r^2 = 0.389$, $P = 0.0010$) and with Commercial Traffic Factor ($r^2 = 0.418$, $P = 0.0004$) (r^2 is the coefficient of determination, P is the probability that r^2 is not zero). A good correlation was also found with climate region ($r^2 = 0.425$, $P = 0.0002$).

In fact, there are reasonably good correlations between rut depth and a number of single factors:

Variable	r^2	P value
Construction Marshall	-0.332	0.0123
Construction Voids	0.282	0.0372
Aggregate passing 40M	0.299	0.0113
Fractured faces	-0.298	0.0115

There is a great deal of hesitancy in putting much emphasis on the correlations with construction Marshall stability and construction voids. These data are based on averages for a paving project of values for stability and voids obtained in the field testing lab for grab samples of the hot mix. Some of the averages were calculated (more recent paving projects) others were merely estimated by scanning the records, since time did not permit calculating the averages. This, combined with the wide variability in the test itself, makes these values questionable.

Among variables which might be expected to be related to rutting, no direct correlations were found with core flow, core asphalt content, core voids, aggregate passing 200M, penetrations, viscosities, PVN, ductility, GPC area percentages, penetration ratio, or viscosity temperature susceptibility. However, as will be seen in Section III.C, interrelationships among variables will cause some of these to be found in the model. This data simply confirms that rutting is a complex phenomenon with contributions from a number of sources.

Marshall Test

The Marshall Test for stability and flow is a widely used pavement design tool. Maximum stability is desired and the amount of asphalt and, possibly, additive in the mix design are varied to achieve that end.

Three values for Marshall stability were obtained for each sampling site. First was Design Marshall, the value derived from the design process described above. Design Marshall was available for 50 sites and the values are reasonably well correlated with values for "Construction Marshall" (correlation coefficient, $r^2 = 0.460$, P-value = 0.0008).

There are also "Core Marshall" values for the samples. They were obtained on individual lifts from cores #2 and #5 at each site. The Marshall Test was designed for use on laboratory-prepared specimens of uniform thickness. Its use on lifts from roadway cores of varying thicknesses requires the use of correction factors. However, this is an established procedure.

Regression tests were performed to compare the Marshall Test results with other variables. Construction Marshall shows no correlation with core Marshall or with asphalt content reported at construction or with most other variables. Stronger correlations (at least for a data set of this nature) were obtained with the following variables:

Construction Marshall vs.

Aggregate, minus 200	$r^2 = 0.411$, $P = 0.0018$
% Fractured faces	$r^2 = 0.498$, $P = 0.0001$
Rut depth	$r^2 = 0.332$, $P = 0.0123$

Core Marshall yielded the following correlations:

Core voids	$r^2 = -0.334$, $P = 0.0039$
Aggregate passing 80 M	$r^2 = 0.328$, $P = 0.0052$
Absolute viscosity	$r^2 = 0.319$, $P = 0.0059$
Ductility 77°	$r^2 = -0.271$, $P = 0.0203$

Means, standard deviations and ranges for Marshall stabilities are of interest.

Variable	Mean	Std. Dev.	Min-Max
Design Marshall	1651.3	650.6	702-3742
Construction Marshall	1341.5	364.8	706-2244
Core Marshall	1320.5	482.8	520-3220

Means for construction and core Marshalls (keeping in mind the difficulties perceived for Construction Marshall data) are similar, although the range of stabilities for core samples is broader. Samples taken during the Pilot Study have Marshall values with a mean of 1600 and a range of 1242-1957.

Values for Marshall flow were also obtained. In the standard procedure for the Marshall test, no correction for varying sample thickness is applied in obtaining flow values.

However, Webb, et. al. (7) recently published a paper documenting the need for such correction factors and proposing flow correlation ratios for samples between 2 and 3 inches thick. It was decided to use these correlation ratios for the data from the rutting study. Some extrapolation was necessary for samples less than 2 or more than 3 inches thick, although this was not extensive.

One might expect that Marshall flow would correlate to some degree with rutting. This data set, however, shows no relationship between rutting and either construction flow or core flow. Statistically, core flow is lower for samples from Region 4 ($r^2 = 0.294$, D = 0.0117) than for other Regions. There is a small correlation of core flow with % fractured faces ($r^2 = 0.283$, P = 0.0168). This would indicate that the Marshall flow increases as the percentage of fractured material increases. There are also correlations of flow with percent aggregate retained on the 10 Mesh screen (passing the 3/8 inch screen) ($r^2 = 0.346$, P = 0.0032) and amount retained on 80 Mesh (passing the 40M) ($r^2 = -0.286$, P 0.00157).

For the non-overlayed data set:

Variable	Mean	Std. Dev.	Min.-Max.
Design flow	10.50	2.79	6.0-15.8
Construction flow	9.54	2.20	6.0-16.0
Core flow	9.77	2.34	4.0-16.0

On average, it would appear that the value of Marshall Flow must not change much with time. However, there is no correlation between design flow or construction flow and core flow, indicating that individual values for flow do change in some more complex fashion.

Voids

There are three values for percent voids available for these sample sites: design voids, that amount derived from the mix design process; construction voids, a calculated or estimated average of voids from the field laboratory testing of mix grab samples; core voids, determined from bulk and Rice specific gravity measurements on cores.

Core voids might be expected to be related to asphalt content but $r^2 = -0.042$, $P = 0.7260$ for this correlation. Core voids do correlate rather well with construction voids, in spite of the caveat about use of this construction data ($r^2 = 0.423$, $P = 0.0013$).

Core voids are correlated with the penetration at 77° of the recovered asphalt ($r^2 = -0.290$, $P = 0.0129$) as well as with the Kinematic viscosity ($r^2 = 0.318$, $P = 0.0060$). There is also a correlation with core Marshall stability ($r^2 = -0.334$, $P = 0.0039$). None of these is particularly surprising.

The correlations with penetration (only that at 77°F, not 40° or 90°F) and viscosity (only at 275°F, not 140°F) are puzzling. These asphalts began service life at a variety of grades; samples were taken from sites paved with 85-100, 100-120,

120-150 and 150-200 grade asphalts from four refineries. Unfortunately, exact original penetrations and viscosities are not known, but the range was obviously wide. That these values have now settled into a pattern that is partially associated with voids content may be explained by the idea that the compactability of the mix during pavement construction is influenced by the viscosity or penetration of the asphalt cement (after heating and mixing with aggregate).

Examination of the means and ranges for the various void measurements is instructive.

Variable	Mean	Std. Dev.	Min.-Max.
Design voids	4.61	1.45	1.68-8.00
Construction voids	4.79	1.36	3.00-8.20
Core voids	5.10	2.62	0.00-13.1
Pilot study core voids	5.32		1.30- 9.83

If it is assumed that the void content of the shoulder does not change appreciably with time because of lack of traffic action, it would appear that the percent air voids is often considerably higher than desired. Also recall, however, that core voids are not directly correlated with rut depth.

Viscosities

As mentioned earlier, the asphalt cements in these samples began their service lives at a variety of penetrations and viscosities, values for which, unfortunately, are not known.

Kinematic viscosity (275°F) is, as might be expected, well correlated with absolute viscosity (140°F) as well as with penetration.

variable		r^2	P
penetration	- 40°F	-0.704	0.0001
	- 77°F	-0.783	0.0001
	- 90°F	-0.811	0.0001
absolute viscosity		0.835	0.0001
ductility	- 77°F	-0.705	0.0001

Absolute viscosity shows correlations with these parameters, as well, but they are not so strong.

variable		r^2	P
penetration	40°F	-0.596	0.0001
	77°F	-0.586	0.0001
	90°F	-0.657	0.0001
ductility	77°F	-0.667	0.0001

Neither viscosity measurement is directly correlated with rut depth.

Kinematic viscosity has small correlations with GPC area percentages (for LMS, $r^2 = 0.248$, $P = 0.0358$; for SMS, $r^2 = -0.232$, $P = 0.0503$). There is also an apparent correlation with core voids ($r^2 = 0.318$, $P = 0.0060$). The latter is more difficult to explain, given the differences in the starting materials.

Absolute viscosity does not have parallel correlations with GPC or core voids, but does correlate with core Marshall stability ($r^2 = 0.319$, $P = 0.0059$).

Values of kinematic viscosity range from 222 to 845 centistokes with a mean at 401. The range of values of absolute viscosity were from 762 to 30,060 with a mean of 3843. In the Pilot Study the mean of absolute viscosity values was 2360 with a range of 1111 to 3149; kinematic viscosity ranged from 281 to 455 and a mean at 380.

Penetration

The penetrations at 40, 77 and 90°F are strongly correlated among themselves and with the viscosities. Pen 77°F is related to ductility ($r^2 = 0.564$, $P = 0.0001$). However, none of the penetrations are correlated directly with rut depth. The means and ranges of values for penetration are:

Pen	Mean	Range
40°F	31 ddm	9-65
77°F	65	22-176
90°F	122	43-216

Ductility

As mentioned earlier, ductility appears to be correlated with core Marshall stability, all penetration values and both viscosity measurements. This is interesting but must be viewed with caution. Because of equipment limitations, it was not possible to perform ductility measurements at lower temperatures (40°F had been proposed). At 77°F, more than 50% of the samples tested were elongated to 150 cm (the limit of the ductilometer) without breaking. It is felt that this compromises the data, and as with construction Marshall stability, there is a great deal of hesitation about emphasizing these correlations to any degree.

Asphalt Content

The amount of asphalt cement to be used in a mix is determined in the design process and, it is hoped, carried out in the construction process. Determination of the latter is made by measuring the amount of asphalt cement used and dividing that by the amount of aggregate used, thus giving an average value for a day's paving.

Extracting asphalt from the mix is, unfortunately, not a simple process especially when a quantitative measure of the amount of asphalt cement in a given sample is desired in addition to recovering the binder for use in other tests. The choice of solvent is important both for its efficiency in removing all the asphalt from the aggregate and for its ease of removal from the asphalt later. Trichloroethylene is traditionally used and was

used here. Care must be taken in the removal of fine aggregate from the binder.

The amount of asphalt cement in the samples was found to correlate somewhat with the design asphalt content ($r^2 = 0.533$, $P = 0.0001$) and with construction asphalt content ($r^2 = 0.588$, $P = 0.0001$). Design and construction asphalt contents were correlated with each other ($r^2 = 0.801$, $P = 0.0001$). This probably indicates that the overall construction process is fairly accurate in producing the desired concentration, but from point to point within the project, larger variation must exist. Indeed in the Pilot Study this was found to be the case. In one project, the percent asphalt was found to range from 5.9 to 8.4 whereas in another the range was much narrower (5.5 to 5.9).

Some interesting correlations were found with asphalt cement content in the cores for this data set. For example, there is a rather strong correlation of core asphalt content with design Marshall stability ($r^2 = 0.453$, $P = 0.0005$) but a lesser one with core Marshall stability ($r^2 = 0.243$, $P = 0.0381$). There is a relationship with design voids content ($r^2 = 0.331$, $P = 0.0135$) but none with core voids percentage. A correlation is also shown with climate zone ($r^2 = -0.309$, $P = 0.0079$) indicating that, on average, pavements in Region 1 may have a higher asphalt content than those in Region 4. Nevertheless, there is no direct correlation with rut depth ($r^2 = 0.092$, $P = 0.4392$).

Asphalt Temperature Susceptibility

Three different approaches to assessing the temperature susceptibility of the recovered asphalt cements were tried. These were penetration viscosity number (PVN), penetration index (PI) and viscosity temperature susceptibility (VTS). None of these variables is correlated directly with rut depth. Furthermore, they are not correlated among themselves.

Both PVN and PI are related to Kinematic viscosity. PVN is calculated from K viscosity, of course. For PI, the $r^2 = 0.276$, $P = 0.0180$. HP-GPC area percentages are related to PVN:

PVN: % LMS $r^2 = 0.538$, $P = 0.0001$

% MMS $r^2 = -0.405$, $P = 0.0004$

% SMS $r^2 = -0.496$, $P = 0.0001$

This means that, as LMS percentage increases (or % SMS decreases), PVN becomes less negative, indicating lower temperature susceptibility. This corroborates earlier observations that certain low LMS asphalts are also highly temperature susceptible. The other measures of temperature susceptibility are not related to molecular size distribution.

Molecular Size Distribution

Molecular size distributions, obtained from the HP-GPC analyses of asphalts, were determined for the recovered binders. Area percentages, particularly LMS (large molecular size) and SMS (small molecular size) do not correlate directly with rut depth.

The only variable showing a significant relationship with HP-GPC areas is PVN, as mentioned above, and K viscosity, also discussed earlier.

Ranges and means for the three GPC fractions are:

% LMS - range	14.9-33.9, mean	22.3
% MMS	38.3-47.7	43.2
% SMS	27.8-41.3	36.5

Aggregate Characteristics

Sieve Analyses

Evaluating the role of aggregate in rutting is a somewhat complicated task. In this research, aggregate recovered from cores was subjected to sieve analyses, using sieve sizes used by MDOH (1", 0.75", 0.5", 0.375", 4M, 10M, 40M, 80M, 200M). Aggregate pieces with one or more fractured faces were counted for material retained on sieve 4M and larger. As noted in the procedure (see Appendix B), cut faces were distinguished from fractured faces by staining all cut surfaces of the core before extraction.

Sieve analyses may be handled in at least two ways, but the method of choice at this time is the 0.45 power chart (Figure 5). Unfortunately, it is difficult to treat these statistically. Therefore, two sets of values were used in the statistical analysis - the percent of aggregate retained on each sieve but

passing the next larger screen, and the percent passing each sieve.

Some interesting relationships have emerged for this data set. For example, the percentage of aggregate passing the 4M screen is correlated with design Marshall Stability (r^2 = 0.272, P = 0.0466) and design asphalt content (r^2 = 0.275, P = 0.0401). On the other hand design Marshall Stability (but not design asphalt content) is related to the amount retained on both 80M (r^2 = 0.273, P = 0.0458) and 10M screens (r^2 = 0.283, P = 0.0383).

In the aged roadway cores, however, the Marshall stability is more closely related to the percentage of aggregate passing the 80M screen (r^2 = 0.328, P = 0.0052) and that retained on the 200M (r^2 = 0.243, P = 0.0414). There are no direct relationships of any aggregate gradation variable with the extent of rutting. Caution must be used in interpreting this data, however, since changes in the amount retained on or passing through a given sieve size are reflected in other percentages.

<u>Fractured faces</u>

The percentage of fractured faces (4M and larger) is correlated with the following variables:

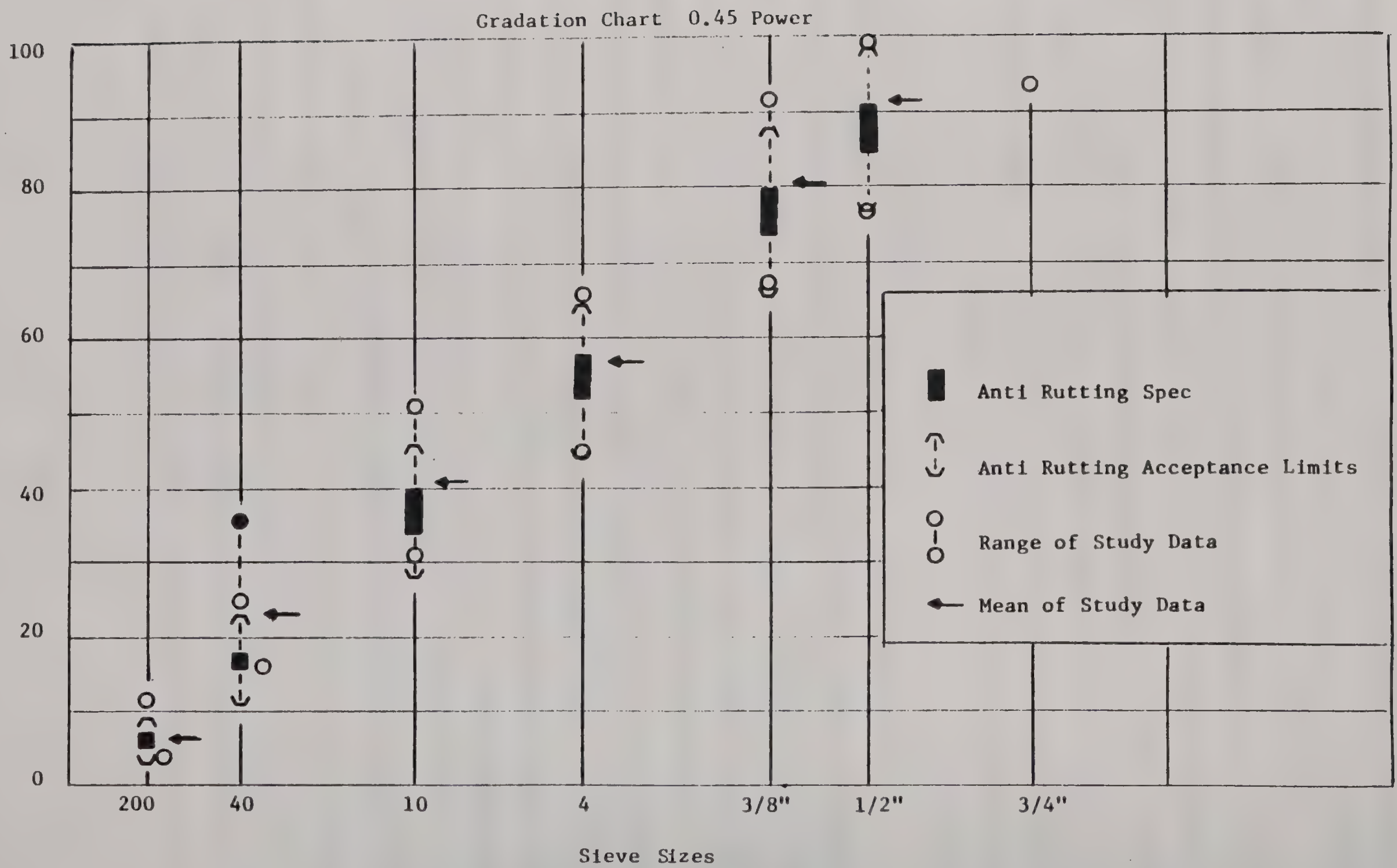


Figure 5. Comparison of Study Aggregate Data with Antirutting Specification

design Marshall stability (r^2 = 0.424, P = 0.0014)

design Marshall flow (r^2 = 0.344, P = 0.0116)

core Marshall flow (r^2 = 0.283, P = 0.0168)

rut depth (r^2 = -0.298, P = 0.0115)

core voids (r^2 = 0.254, P = 0.0324).

Correlation with design Marshall stability is not unexpected, although there is no concommitant relationship with core stability.

The association of fractured faces and ruts is also not unexpected. With regard to the increases in core voids with increased fractured faces, perhaps the presence of fractured aggregates interferes with compaction to some degree.

We are not presently satisfied with the statistical approach to the influence of aggregate on the rutting problem and plan to continue work in this area. However, a compilation of the present data set may be useful. In Figure 5 is plotted the antirutting specification for aggregate gradation recently adopted by MDOH along with ranges and means of the samples in the data set. It can be seen that, in general, the extremes for each sieve size are within, or very close to, the acceptance limits of the specification. The means are, generally, within or a little higher than the specification range. The one screen size that shows the most serious discrepancy with the anti-rutting specification is the 40 M. Notice, however, that the mean for those values is pulled higher by the existence of <u>one</u> sample with

nearly 35% passing the screen. Otherwise, most of the samples would fit within the acceptance limits. It goes without saying that, in spite of the fact that most of the aggregates would satisfy the anti-rutting specification limits, a great deal of rutting has been observed in the pavements sampled.

Climate Region

As discussed earlier, a means of assessing the influence of climate, particularly of hot summer temperatures, was desired. Figure 6 is a sketch map of the Interstate system with Climate Region numbers coded along the routes. Note that most of the roadway is in either Region 3 or 4. Very few sampling sites are located in Regions 1 and 2.

Region 4 is very significant in the statistical analysis of rutting, as will be seen in the next section. Region 4 differs from other Regions in that its average August temperatures are higher. It also coincides in part with the area wherein aggregates are frequently obtained from the Yellowstone River gravels. In spite of the fact that most aggregate gradations sampled fit within the requirements for the anti-rutting specification, as discussed above, the question remains as to whether the aggregates found in Region 4 samples are significantly different from those in other Regions. Data in the following table can help to answer that question.

Table 3 shows that there appears to be a trend toward decreasing amounts retained on the 10 M and 40 M screens and increasing amounts retained on the 80 M and 200 M screens as the

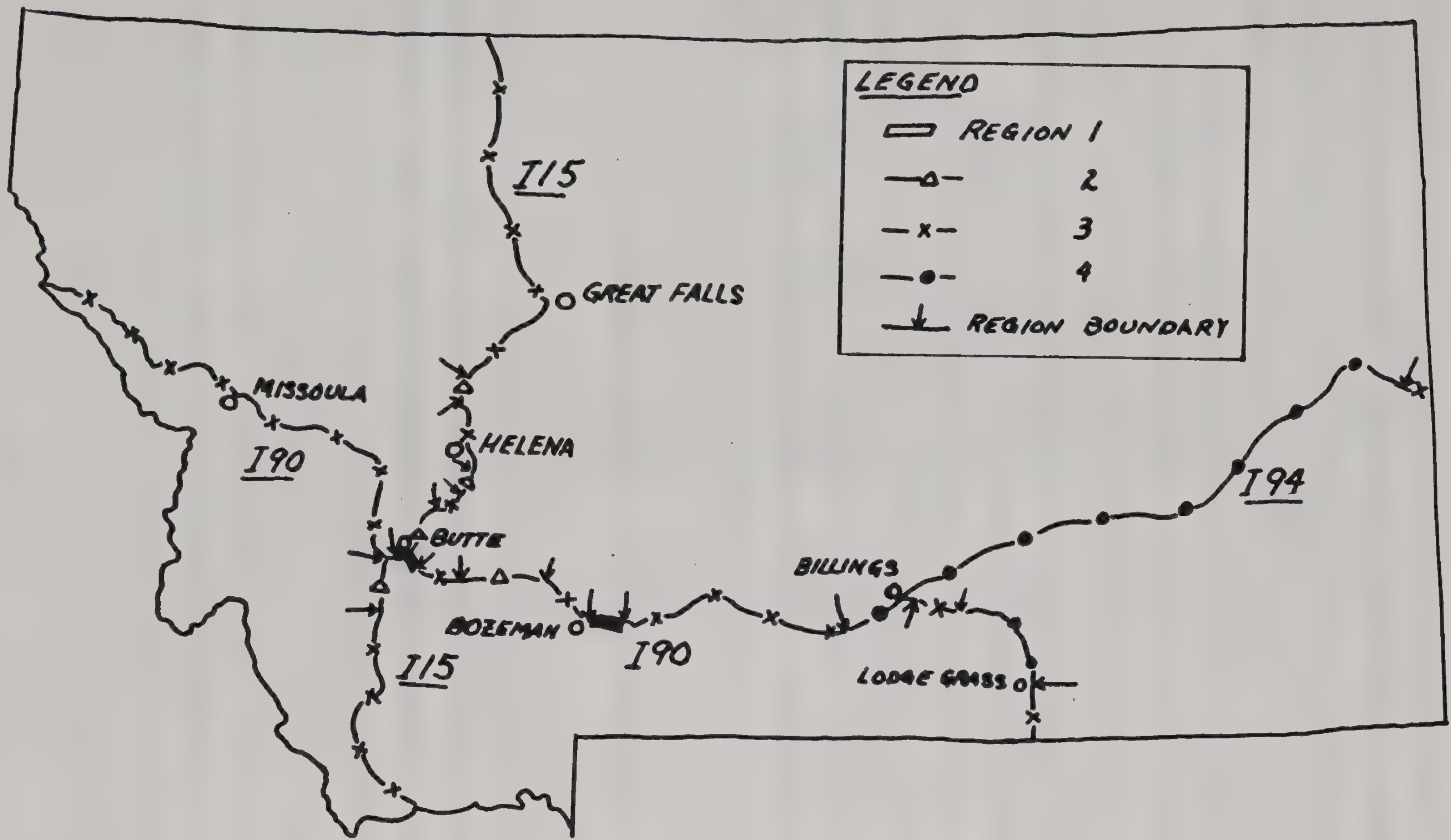


Figure 6. Sketch map of Climate Regions on Interstate Highways

climate region number increases. This, of course, implies no relationship with climate but rather is coincidental. Also, there appears to be a significant difference between regions 3 and 4. Therefore, even though aggregate gradations as presently handled in the statistical analysis do not make a large contribution to the rutting model, as will presently be seen, they may have an influence under the guise of Climate Region. Again, some caution is advised because of the small number of observations in Regions 1 and 2.

Similarly, there is a trend toward lower percentage of fractured faces in Region 4 as well as a trend toward higher commercial traffic factor. Therefore, it seems likely that the influence of Region 4 in the model may derive from not only climate, but also aggregate factors and traffic factors as well.

C. Statistical Model for Non-Overlay Pavements

Background

Before presenting the model which has been derived from the data, it is necessary to discuss the process by which the model was derived. Some of the points have been made earlier but will be reiterated here for clarity.

Table 3. Percent of Aggregate Retained vs. Climate Region

Screen Size	N	Minimum	Maximum	Mean	Std Dev
REGION 1					
AG10	2	18.00	19.20	18.60	0.85
AG40	2	16.70	19.40	18.05	1.91
AG80	2	9.60	10.50	10.05	0.64
AG200	2	5.30	5.60	5.45	0.21
REGION 2					
AG10	8	12.70	26.20	17.27	4.34
AG40	8	19.90	25.40	22.00	1.93
AG80	8	3.50	11.20	7.17	2.34
AG200	8	3.80	6.90	5.14	1.12
REGION 3					
AG10	39	9.60	24.30	16.33	2.82
AG40	39	9.20	27.60	18.54	4.65
AG80	39	4.20	18.20	10.33	2.69
AG200	39	2.80	7.30	5.56	1.22
REGION 4					
AG10	21	7.10	21.90	15.07	3.78
AG40	21	8.20	21.20	13.75	3.66
AG80	21	8.10	23.20	13.61	3.90
AG200	21	4.90	10.20	7.32	1.51

The full data set contained rutting data for sample sites ranging in age from 0 to 25 years with rut depths measuring between 0 and 31 mm. This data is categorized in Figure 4, pg 20. Initially, an effort was made to construct a simple, stable model for all roads in the data set. However, no meaningful models were obtained. In this early work, two variables appeared to be especially significant: commercial traffic factor (commercial traffic in 1985 x pavement age) and construction Marshall stability. However, when efforts were made to replace

construction Marshall stability with variables which contribute to it (eg., percent fractured faces, core voids, asphalt content) the models became either uninformative or unstable. Therefore, construction Marshall stability was eliminated from the list of possible predictors.

Models for the full data set remained unsatisfactory, however, and upon further examination it was noticed that younger roads appeared to rut at a different rate than older roads. Although there is evidence that the most severe rutting may occur in the first years after construction, further examination showed that the problems with modeling this data set arose not so much from differences in rutting rate as differences in construction. That is, pavements that contained overlays could not be directly compared with pavements which had not been overlayed. Therefore, efforts focused on non-overlayed pavements. The number of pavements in categories of rut depth by age are shown in Figure 7.

Age	Ruts,mm <12.7	12.7-19.0	>19 ruts
0-5	14	0	0
6-10	8	5	0
11-	23	17	5

Figure 7. Distribution of sites for non-overlayed pavements

The contributions of age and commercial traffic factor were acknowledged by multiplying age by the variables and commercial traffic factor by the variables and adding the resulting factors to the list of possible predictors. Rationale for this is that some variables might contribute an increment of rutting per year or per commercial traffic load.

Ductility x age was a significant term in earlier versions of the model. However, because perhaps 64 percent of the samples in the non-overlayed set had ductilities of 150 cm or more (150 cm is the limit of measurement) it was decided to delete ductility from the list of predictors.

Rut depths used in the final model consisted of the average of three measurements made in the most severely rutted wheelpath. This value was selected as being of more value to MDOH. Use of the average of rut depths measured in both wheelpaths of the driving lane would not likely have changed the final model in terms of predictors and coefficients, but the r^2 value may have been improved.

Climate region 4 has been a significant predictor in all the modeling attempts, including those with full data set as well as those with non-overlayed pavements only. Region 4 was always significantly different with regard to ruts. No statistically significant difference was found among regions 1, 2 and 3 in that respect. This trend is diagrammed in Figure 8. (It should be noted that there were few observations in regions 1 and 2.)

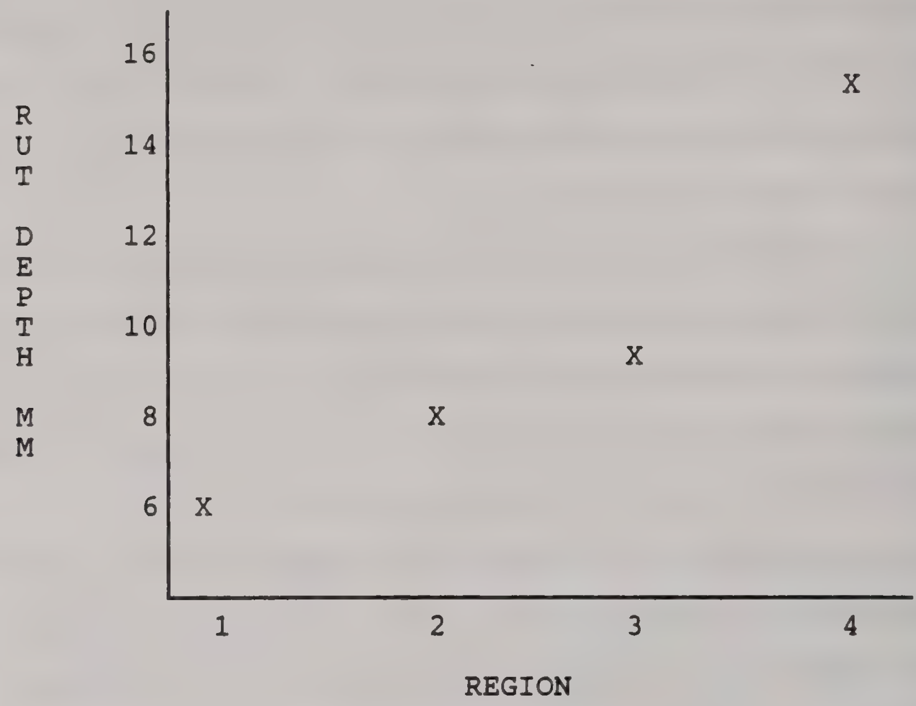


Figure 8. Average rut depth in climate regions

Several approaches were used to include aggregate gradation data in the list of predictors. Percentage of aggregate passing each sieve as well as the percentage retained on each screen (and passing the next larger screen) were included. Also considered was a classification system in which percent "course" aggregate included material retained on 4 mesh and larger; "fines 1" the percentage on screens 10 and 40 mesh; and "fines 2", the percentage on screens 80, 200, and minus 200. In addition, a variation of the maximum models measurement used for Portland cement concrete aggregate was considered. Of these choices, the percent retained on certain screen sizes were found to contribute

to a rutting model. A convenient way to use 0.45 power curves was not immediately available and further effort was not devoted to finding a way to use them at this time.

A number of categorical variables, including refinery source, asphalt cement grade, additive or filler type, mix plant type, etc., were available. Differences could not be found between these terms by themselves, at least partly because of age. For example, the grade of asphalt cement used has changed with time largely because of design philosophy changes at MDOH. Now that a model has been constructed, it may be possible to detect differences in these variables through an ACOVA analysis. Time did not permit this, however.

In the statistical modeling, data from the uppermost lift of the shoulder gave better results than did data from the wheelpath of the driving lane. Difference in voids content between shoulder and wheelpath was utilized but did not enhance the model. Therefore, shoulder data was used for modeling. It is thought that the shoulder best represents the pavement condition at the time of construction. This may be true even for penetrations and viscosities. It is known that viscosity tends to increase with age depending on depth within the core, at least in some geographical areas. That is, asphalt near the surface undergoes a greater increase in viscosity with age than does that deeper in the pavement. At lower positions, the viscosity remains very similar to the value at construction (after mixing and laydown). Experience in this laboratory to date shows that

the molecular size distribution of an asphalt undergoes its most drastic change during mixing with aggregate and thereafter in Montana climates, does not change appreciably. Since surface portions of each core sample were discarded in the recovery of asphalt cement for penetration and viscosity testing as well as for HP-GPC analysis, it may be that penetration and viscosity values have not changed very much with time. It should be noted, however, that often, although not always, values of penetration were lower and viscosities higher for asphalts recovered from the shoulder than from the driving lane. No doubt this subject requires more study.

The Model

Pertinent statistical data for the model for rutting for the data set consisting of non-overlayed pavements in the Montana Interstate Highway system is contained in Table 4.

In general, this model states that Region 4 contributes 4.51 mm of rutting; that higher percentages of fractured faces, larger percentages of aggregate retained on the 10 M screen and higher values of absolute viscosity reduce the amount of rutting (signified by their negative parameters); that higher voids percentages, higher asphalt contents, higher percentages of minus 200 aggregate, higher penetration at 77°F, and higher values of kinematic viscosity result in increases in the amount of rutting.

Table 4. Model for Rutting

Variable	Parameter[9] Estimate	Std.[10] Error	Probability[11]
Intercept	-4.11	5.53	0.4603
Faces[1]	-1.63×10^{-5}	3.0×10^{-6}	0.0001
Region 4	4.51	1.03	0.0001
Agg. 10[2]	-0.45	0.16	0.0061
Voids[3]	0.036	0.02	0.0512
%Asphalt[4]	1.50	0.82	0.0728
Minus 200[5]	1.02	0.39	0.0106
Pen 77[6]	5.61×10^{-3}	1.09×10^{-3}	0.0001
A. vis[7]	-6.65×10^{-5}	1.89×10^{-5}	0.0009
K. vis[8]	3.88×10^{-3}	6.32×10^{-4}	0.0001

F Value 10.989

Adjusted R^2 0.5880

No. of observations 64

(1) Percent fractured faces x commercial traffic factor.

(2) Percent of aggregate retained on 10 M screen.

(3) Age of pavement x percent voids.

(4) Asphalt content by difference method.

(5) Percent of aggregate passing the 200 M screen.

(6) Penetration of recovered asphalt at 77°F.

(7) Age of pavement x absolute viscosity (140°F) of recovered asphalt.

(8) Commercial traffic factor x kinematic viscosity (275°F) of recovered asphalt cement.

(9) The amount by which the value of the variable is multiplied in the equation for the model.

(10) The standard error for the parameter estimate.

(11) An expression of the likelihood that the parameter is actually zero. A value of 0.0500 or less is most desirable (ie, the probability is 5 percent or less that the parameter value is zero.)

Only the last of these statements is illogical. It does not seem reasonable that an increase in viscosity would result in an increase in rutting. However, none of these variables exists in a vacuum. The apparent contradiction between the effects of absolute and kinematic viscosities probably results from an interaction between the two measurements, an indication of temperature susceptibility, perhaps. However, as mentioned earlier, viscosity temperature susceptibility was added to the predictor list but did not enhance the model.

The model is, of course, an equation in which the depth of rutting is the sum of all of the predictors in the model multiplied by their respective parameters. For example, for sites 10 and 91, the contributions of the various factors are given in Table 5.

Table 5. Contributions of Predictors to Predicted Rut Depth

	Site 10	Site 91
Intercept	-4.11	-4.11
-1.63×10^{5} (CTF)[1] (% fractured)	-4.11	-4.32
+4.51 (Region 4)	+0.00[2]	+4.51
-0.45 (Agg. 10)	-8.16	-5.31
+0.036 (Age) (% voids)	+0.87	+0.60
+1.5 (% asphalt)	+8.05	+7.68
+1.02 (minus 200)	+6.65	+5.53
$+5.61\times10^{-3}$ (pen 77)	+1.71	+3.88
-6.65×10^{-5} (Age) (A. vis)	-0.96	-1.62
$+3.88\times10^{-3}$ (CTF) (K. vis)	+7.81	+6.31
Total predicted rutting	7.75 mm	13.15 mm
Measured rutting	7.7	10.0

(1)Commercial traffic factor
(2)Not in Region 4

The actual rut depth for site 10 is 7.7 mm (0.30 in), that for site 91 is 10.0 mm (0.39 in). From the calculations in Table 5 it can be seen that the rutting at site 10 is accurately predicted by the model whereas at site 91, rutting is predicted within about 3 mm (0.12 in). Similar calculations for each of the sites included in the non-overlay data set (given in Appendix C) show that the model predicts the observed rut depth for about 81% of the sites within ±4mm (±0.16 in.) and for 95% of the sites within ±6 mm (±0.24 in.). Only 3 sites have measured rutting that differs by more than 6 mm from that predicted. The difference between rut depth predicted by the model and measured rut depth is the residual for a particular site. Figure 9 contains a plot of the residuals against the predicted rut depths. Dashed lines enclose the region ± 4 mm (± 0.16 in) and

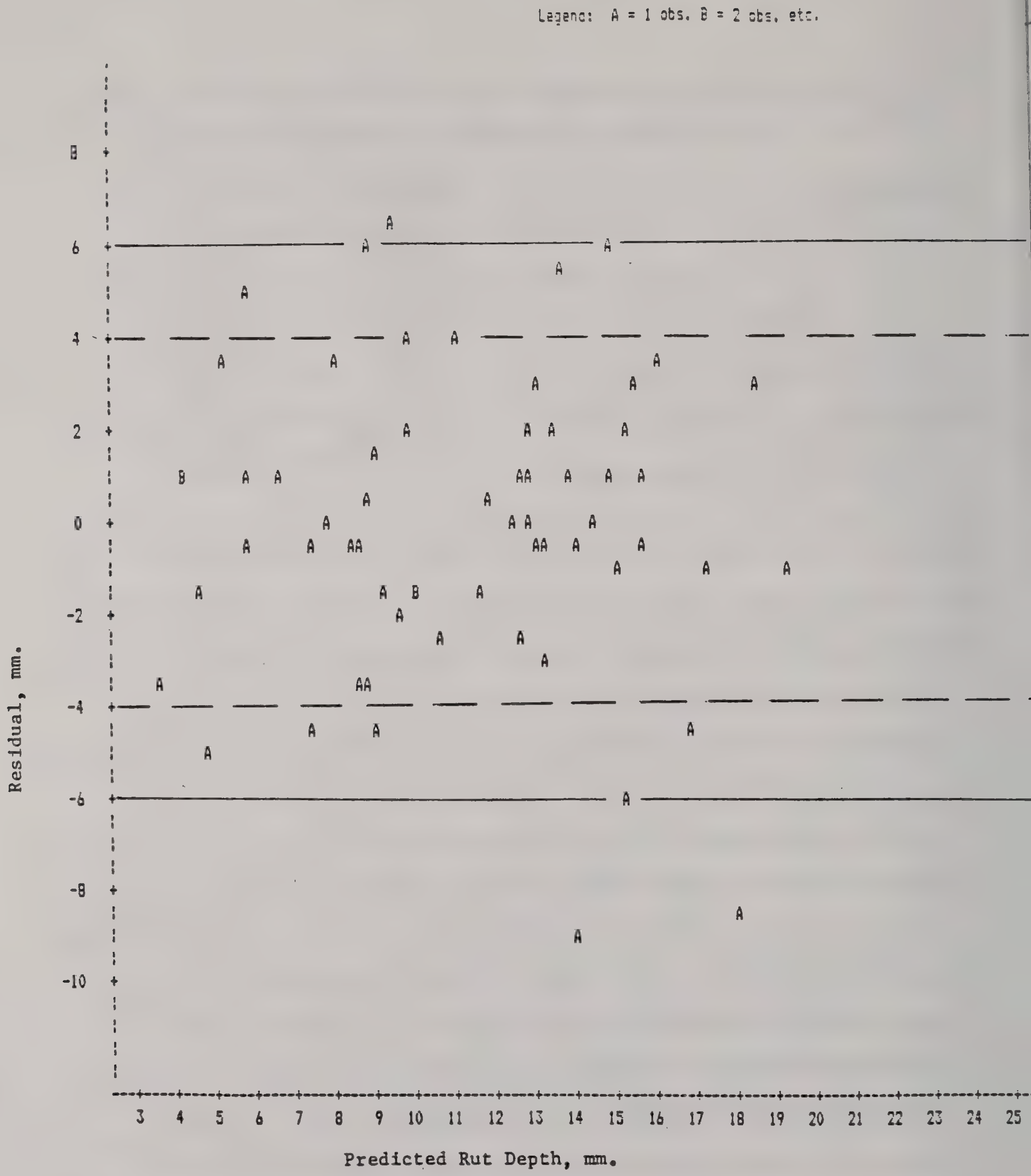


Figure 9. Plot of Residuals Against Predicted Rut Depths

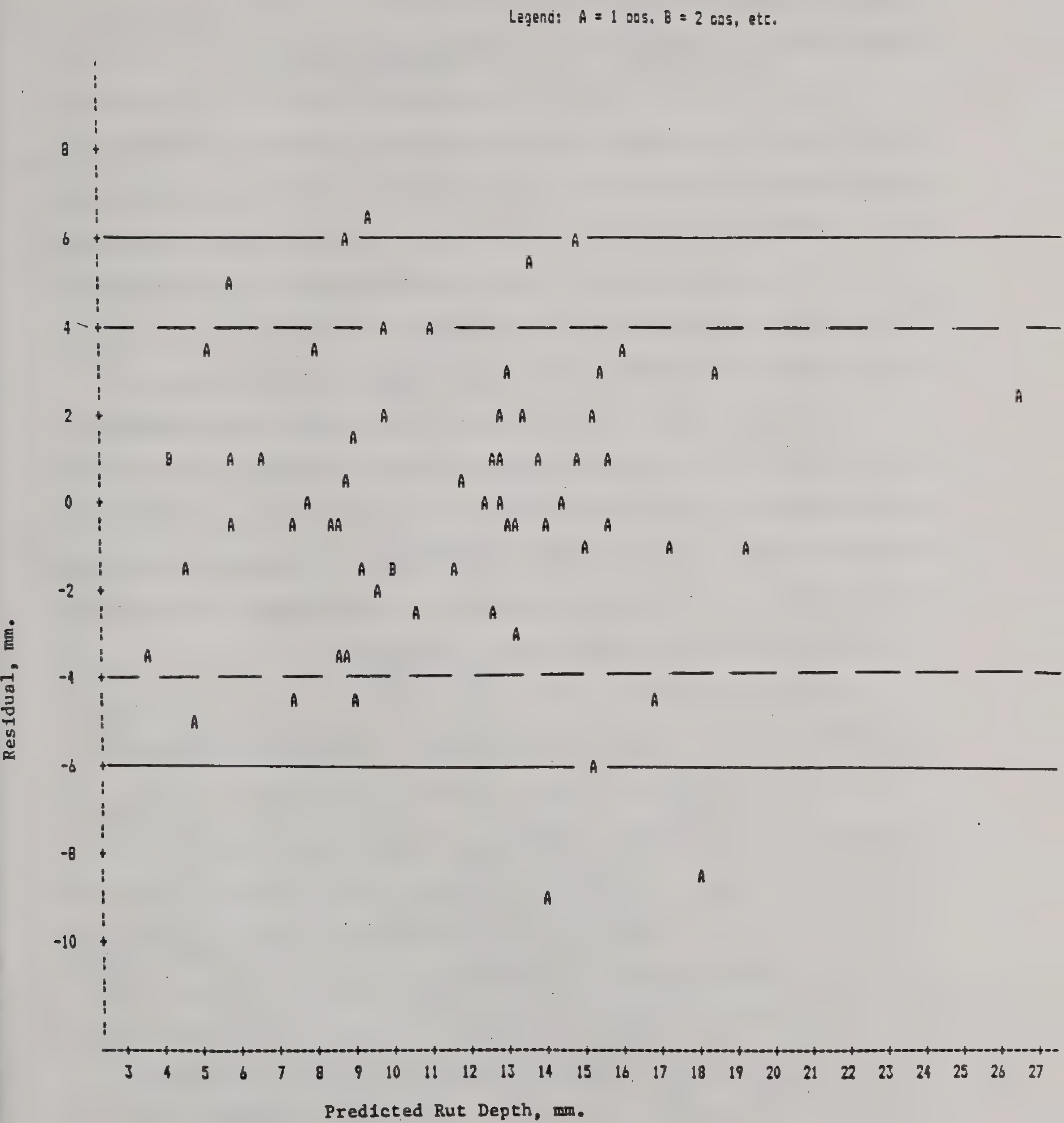


Figure 9. Plot of Residuals Against Predicted Rut Depths

solid lines the region ± 6 mm (± 0.25 in). This is a very reasonable distribution for such a data set.

Ultimately, of course, the goal is to use this statistical information to reduce rutting. A casual look at the model might produce the observation that the percentage of fractured faces, the percentage of aggregate retained on the 10 mesh screen, and the absolute viscosity of the finished asphalt cement must be increased, whereas % voids, % asphalt, % aggregate minus 200, penetration and kinematic viscosity must all be reduced.

However, there are practical limits to the variables in addition to the fact that many of them are interdependent. Nevertheless it is possible to use the non-overlayed data set to estimate the minimum rutting that might be expected from a good combination of predictor variables and conversely, the maximum rutting from a poor combination of factors.

For this exercise, the values selected for the predictor variables are the mean and the mean ±1 standard deviation for that variable actually found in the non-overlayed data set. For example, for aggregate retained on the 10M screen the mean is 16.1%, the standard deviation is 3.3. Thus values of 12.8, 16.1 and 19.4% were multiplied by the coefficient -0.45 to give the expected contributions to rutting for this variable. In Table 6 are the results of such a calculation for a pavement 6 years old with commercial traffic at 690 vehicles per day (commercial traffic factor (CTF) = 6 x 690 = 4140. It is assumed that the pavement is not in Region 4.

Table 6. Calculated Range of Rutting Performance Based on Non-overlayed Data Set for a 6 Year Old Pavement.

Predictor	Coeff.	Range	Min.	Ave.	Max.
Intercept	-4.11	--	-4.11	-4.11	-4.11
Faces	-1.63×10^{-5}	4142(79±12.4)	-6.17	-5.33	-4.50
Region 4	4.51	--	--	--	--
Agg 10	-0.45	(16.1±3.3)	-8.73	-7.25	-5.76
Voids	0.036	6(5.1±2.62)	0.54	1.10	1.67
% Asphalt	1.50	(5.7±0.58)	7.68	8.55	9.42
Minus 200	1.02	(6.5±1.5)	5.10	6.63	8.16
Pen 77	5.61×10^{-3}	6(65±25)	1.34	2.18	3.02
A vis	-6.65×10^{-5}	6(3478±2281)	-2.30	-1.39	-0.48
K vis	3.88×10^{-3}	(395±100)	4.74	6.35	7.96
Total Rut depth at 6 years, mm			-1.91	6.73	15.38

In Table 7 are the results of similar calculations for the same pavement at 10 and 15 years of age, respectively.

It would appear from these calculations that rutting can be avoided in Montana's Interstate pavements. Indeed, that is not only possible but has been observed in that there are at least four Interstate sections more than 10 years old with rutting of 5 mm or less (See Figure 4). However, caution must be used in interpreting this data. Several assumptions have been made in calculating the expected ruts. The first is that all of the

Table 7. Calculated Range of Rutting Performance Based on Non-overlayed Data Set for pavement at 10 and 15 years.

	Expected Ruts					
	10 years			15 years		
Predictor	Min	Ave	Max	Min	Ave	Max
Intercept	-4.11	-4.11	-4.11	-4.11	-4.11	-4.11
Faces	-10.28	-8.89	-7.49	-15.42	-13.33	-11.24
Region 4	--	--	--	--	--	--
Agg 10	-8.73	-7.25	-5.76	-8.73	-7.25	-5.76
Voids	0.89	1.84	2.78	1.34	2.75	4.17
% Asphalt	7.68	8.55	9.42	7.68	8.55	9.42
Minus 200	5.10	6.63	8.16	5.10	6.63	8.16
Pen 77	2.24	3.65	5.05	3.37	5.47	7.57
A. Vis.	-3.83	-2.31	-0.80	-5.74	-3.47	-1.19
K. Vis	7.90	10.57	13.25	11.85	15.86	19.88
Total,mm	-3.14	8.68	20.50	-4.66	11.10	26.90

predictor variables are independent, which is, of course, not true. One could not, for example, change the penetration of an asphalt without changing its viscosity as well. Another assumption in these calculations is that the volume of commercial traffic is constant. Also, the range that is used for values of each variable is the mean of values obtained for the data set ±1 standard deviation and so does not represent the limits to which a variable might be taken. For example, for percent fractured

faces, the range used in these calculations is 79±12.4. That is, the maximum used is 91.5% whereas the maximum attainable is 100%.

Some of the predictor variables include either age or commercial traffic factors which make their influence more strongly felt with time. Percent fractured faces is one of these. At 6 years, 91% fractured faces contributes -6.2 mm of rutting; at 10 years, -10.3 mm; at 15 years, -15.4 mm. In other words, 91% fractured faces in the aggregate, 4M and larger helps to prevent about 1 mm of rutting per year. On the other hand, percent asphalt and other variables do not require an age or traffic dependency in the model.

D. Statistical Models for Rutting in Overlays

The design of the study, the collection and statistical analysis of data have been described in detail in previous parts of this report. A few points specific to the overlayed pavements will be made here.

There are two groups of overlay-containing pavements, those with open graded friction courses (OGFC) in addition to the dense graded overlay, and those without OGFC. Open graded friction courses are fairly thin layers, and do not lend themselves to the testing done on dense graded pavements. No tests were done on OGFC's during this study. The effect of that is, in essence, a tacit assumption that rutting is controlled by the dense graded pavement layers. However, it was noted on page 22 that, at four sites, badly stripped dense graded lifts were found under OGFC. Rutting at those sites was negligible at the time of sampling (1986) and had not changed significantly when those sites were resurveyed in 1988. In other words, it is not entirely accurate to assume that rutting is controlled by the dense graded pavement.

With this in mind, it was decided to attempt to derive statistical models by grouping the data for overlay pavements in three ways: 1) the whole set including OGFC-containing pavements as well as those without OGFC; 2) pavements with OGFC only; 3) pavements without OGFC only. This has at least one serious disadvantage in that groups 2 and 3 have reduced numbers of observations, making modeling more difficult and less reliable.

In fact, the set containing only overlays without OGFC did prove to be impossible to model. However, the other two data sets did yield reasonable models which will be discussed.

Models for Overlays with OGFC

In this section, two possible models for rutting in overlays with OGFC's will be presented. The differences between them as well as the problems leading to a proliferation of models will be discussed.

Model I. Figure 10 contains a plot of rut depth (in mm) versus the age of the overlays. This shows, first, that all of the pavements in the data set, except one, are 6 years old or less, and most are two years old (37 of 62). This has some ramifications for the statistical work in that a wider spread of ages would be desirable.

Maximum rut depths are just under 0.5 inch. This narrow range is also less than ideal for statistical modeling.

For this modeling effort, the list of independent variables has been essentially the same as for the non-overlayed pavements, except that data for both overlay and original pavements have been included.

Table 8 contains essential information for Model I. There are 9 variables in this model.

It is not surprising that certain variables from the original pavement appear to influence rutting in the overlay. However, it is interesting that penetration at 40 °F and aggregate retained on the 80M screen do not enter the model for

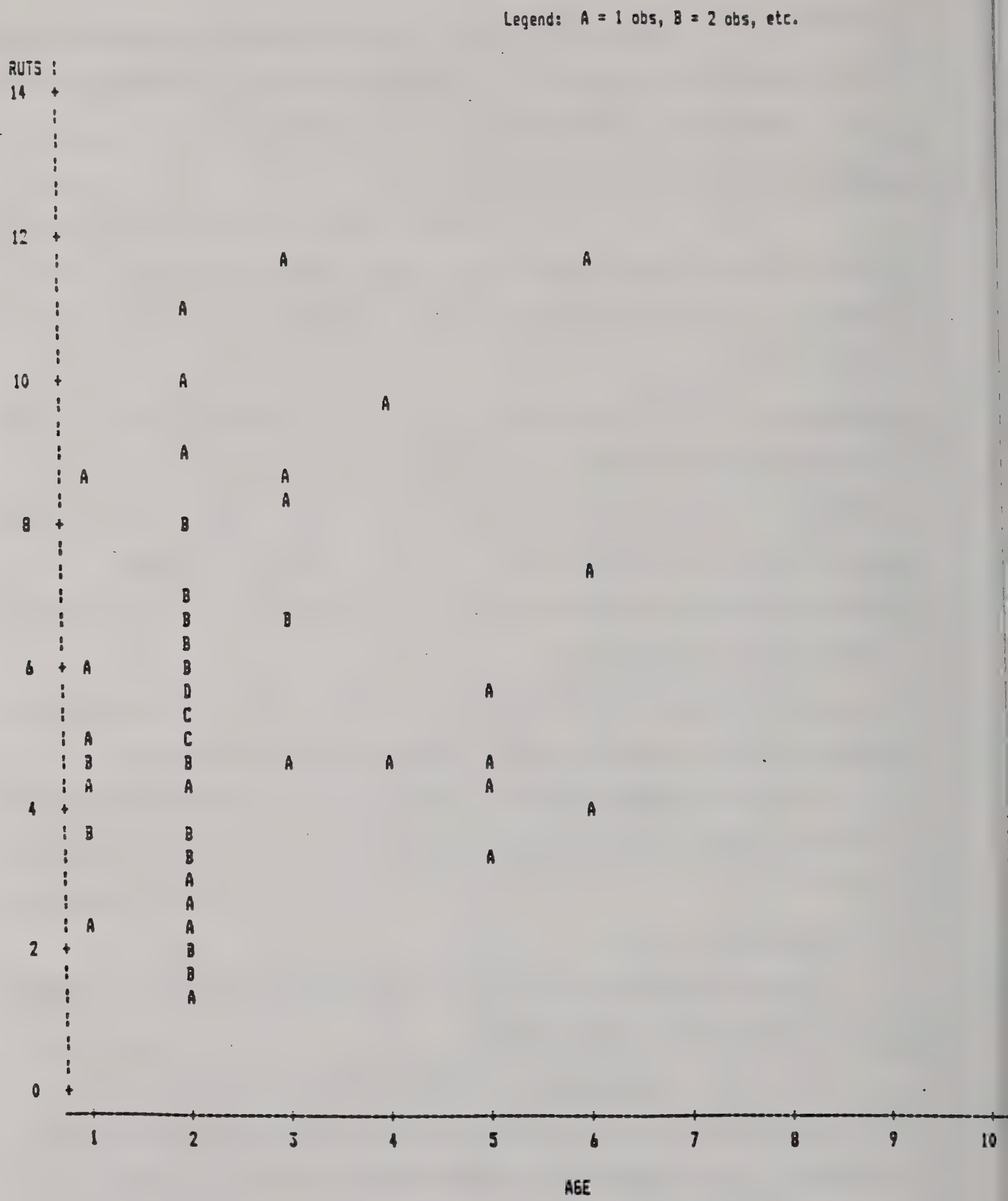


FIGURE 10. Plot of Rut Depths (mm) vs. Pavement Age for OGFC-containing Projects.

non-overlayed pavements. Also note that the sign of the parameter for pen 40 indicates that as pen increases rutting decreases. Considering pen 40 in isolation, that is not logical.

Table 8. Model I for rutting in overlays with OGFC

Variable	Parameter[8] Estimate	Standard[9] Error	Probability[10]
Intercept	-8.54	4.29	0.0528
Faces[1]	-0.08	0.02	0.0003
Agg. 40[2]	0.56	0.10	0.0001
Pen 40[3]	4.88×10^{-5}	9.3×10^{-6}	0.0001
K. vis*	-2.63×10^{-3}	9.1×10^{-4}	0.0059
A. vis	2.31×10^{-4}	1.3×10^{-4}	0.0855
Pen 40*[4]	-1.99×10^{-5}	8.7×10^{-6}	0.0272
Agg. 4[5]	0.43	8.9×10^{-2}	0.0001
% LMS[6]	-0.15	8.3×10^{-2}	0.0823
Agg. 80*[7]	-0.29	8.4×10^{-2}	0.0014

F value 13.564
Adjusted R^2 0.6809
No. of observations 53

*Value of variable in original pavement
(1) Percent fractured faces in overlay
(2) Percent of aggregate retained on 40M after passing 10M
(3) Penetration of recovered asphalt at 40 °F x commercial traffic factor
(4) Penetration at 40 °F of asphalt recovered from original pavement x commercial traffic factor
(5) Percent of aggregate retained on 4M after passing 3/8" screen
(6) Percentage of LMS material in recovered overlay asphalt by HP-GPC
(7) Percent of aggregate in original pavement retained on the 80M after passing the 40M screen
(8) The amount by which the value of the variable is multiplied in the equation for the model
(9) The standard error for the parameter estimate
(10) An expression of the likelihood that the parameter is actually zero. A value of 0.0500 or less is most desirable (i.e., the probability is 5% or less that the parameter value is zero)

Nevertheless, none of these variables is isolated.

Just as in non-overlayed pavements, viscosities make very puzzling contributions to this model. In Model I, kinematic viscosity of the lower lift has a negative sign, i.e., as viscosity increases, rutting decreases as one might expect. Absolute viscosity of the upper lift also enters the model, but with a positive sign. Measures of temperature susceptibility, including PVN, VTS, PI as well as the difference between the viscosity values (after converting to the same units) were put into the list of potential independent variables. However, none of these was found to be significant.

Model II. These seeming contradictions prompted the suggestion that a second model be constructed after removing outliers of both absolute viscosity and penetration at 40 °F of the lower lift from the list of potential variables. When this was done, there were several changes in the Model (See Model II, Table 9). For example, kinematic viscosity dropped from the Model. Also, Model II contains no reference to the percentage of fractured faces.

In Model II, the percentage of minus 200 aggregate enters with a negative parameter. In non-overlayed pavements, the parameter for this variable was positive. This is not easily justified except to say that, for the overlay data set, lower percentages of minus 200 are associated with rutting.

Table 9. Model II for overlay pavements with OGFC, absolute viscosity outliers deleted

Variable	Parameter Estimate	Std Error	Probability
Intercept	-15.15	3.97	0.0004
Agg 40$^{(2)}$	0.50	0.10	0.0001
Minus 200	-0.54	0.16	0.0016
Pen 40$^{(3)}$	5.5×100^{-5}	1.02×10^{-5}	0.0001
Agg 4$^{(2)}$	0.42	0.08	0.0001
*Agg 80$^{(2)}$	0.37	0.07	0.0001
%LMS$^{(4)}$	4.4×10^{-5}	1.62^{-5}	0.0095

F value 17.784
Adjusted R^2 0.6595
Number of observations 53

*Values for original pavement

(2) Percent of aggregate retained on the screen having passed the next larger screen
(3) Penetration at 40 °F x commercial traffic factor
(4) Percent LMS in recovered asphalt x commercial traffic factor

The LMS content is modified by traffic levels in Model II and is more significant (probabilities: 0.0823 in Model I, 0.0095 in Model II).

Plots of residuals, that is the difference between predicted and measured rutting, for these models are in Figures 11 and 12. For Model I (Fig. 11), rut depths on all but 4 of the sites (8%) are predicted within ±2 mm; for model II, the data is similar. Model I predicts 33 sites (62%) within ±1mm; Model II predicts 65% within this range.

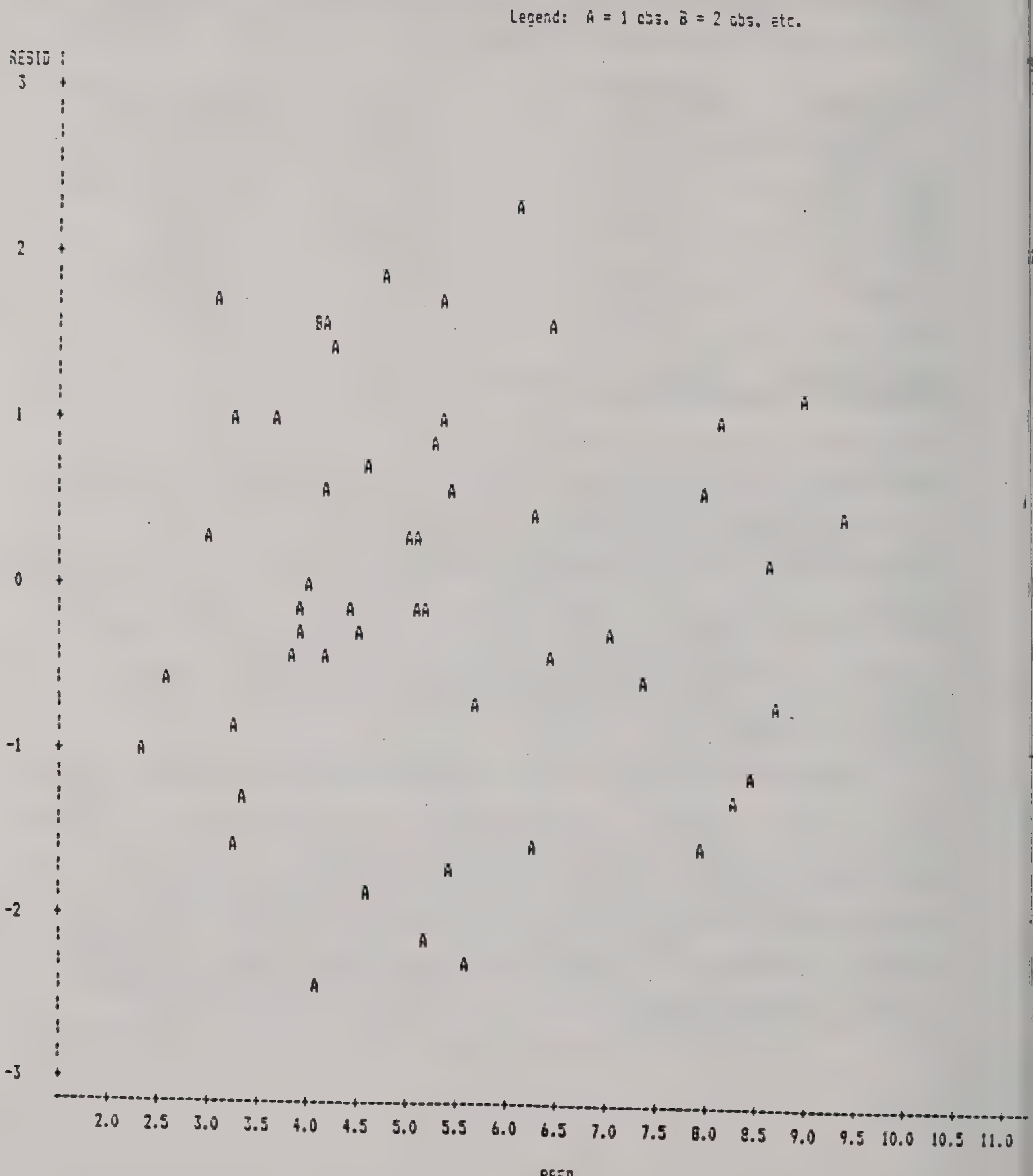


FIGURE 11. Plot of Residuals vs. Predicted Rut Depths (mm) for Model I.

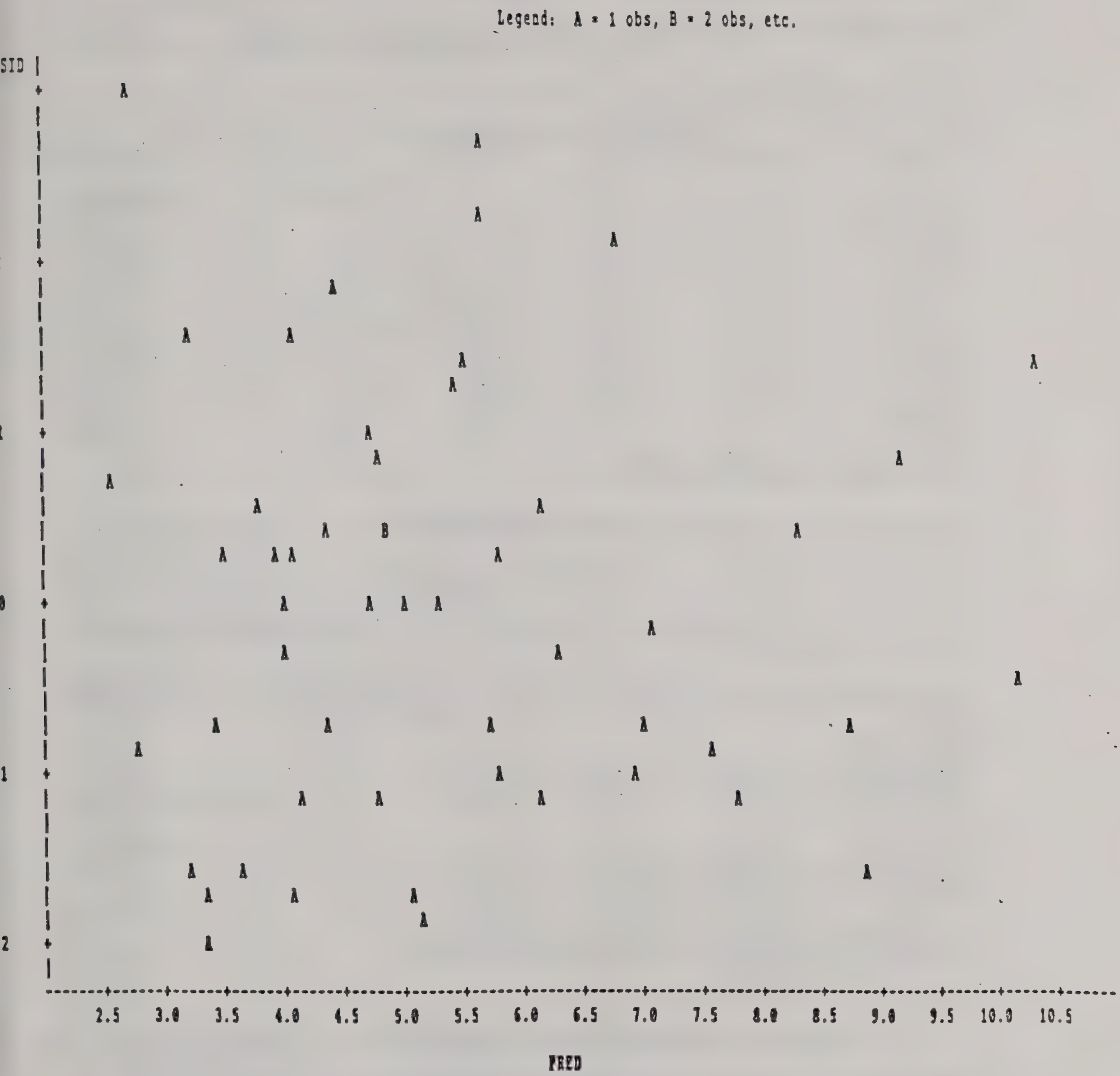


FIGURE 12. Plot of Residuals vs. Predicted Rut Depth (mm) for Model II.

Contributions of the variables in each of the models to rutting in overlays with OGFC are detailed in Tables 10 and 11.

Table 10. Contributions of Predictors to Predicted Rut Depth Model I

	Site 23	Site 184
Intercept	-8.54	-8.54
Faces	-8.47	-6.50
Agg 40	9.45	10.34
Pen 40(1)	11.73	0.77
K. vis*	-2.13	-0.90
A. vis	0.52	1.38
Pen 40*(1)	-3.27	-0.54
Agg 4	13.55	9.76
% LMS	-3.50	-3.70
Agg 80*	1.81	2.04
Predicted rut depth, MM	11.2	4.1
Measured rut depth, MM	11.7	5.7

1) Modified by commercial traffic factor
*original pavement

Table 11. Contributions of Predictors to Predicted Rut Depth - Model II

	Site 23	Site 184
Intercept	-15.15	-15.15
Agg 40	8.75	9.25
Minus 200	-2.67	-3.59
Pen 40(1)	3.57	0.87
Agg 4	10.89	9.51
Agg 80*	2.62	2.62
% LMS(1)	-1.76	-0.79
Predicted rut depth, mm	10.3	2.7
Measured rut depth, mm	11.7	5.7

(1) Modified by commercial traffic factor
*original pavement

The potential range of contributions from the predictors for each model based on the values of the mean and the mean ± 1 standard deviation of the variable actually found in the data set are given in Tables 12 & 13.

Table 12. Calculated Range of Rutting Performance Based on Overlay plus OGFC Data Set - Model I

Predictor	Coeff.	Approx. Range	Theoretical Rut Depth, mm Min.(1)	Ave.(2)	Max(3)
Intercept	-8.54	--	-8.54	-8.54	-8.54
Faces	-0.08	(86 ± 9)	-7.60	-6.88	-6.16
Agg. 40	0.56	(18 ± 3)	+8.4	+10.08	+11.76
Pen 40(4)	$4.88x10^{-5}$	(29 ± 8)	+0.36	+2.73	+5.09
K vis*	$-2.63x10^{-3}$	(614±330)	-2.48	-1.61	-0.75
A vis	$2.31x10^{-4}$	(3980±2589)	+0.32	+0.92	+1.52
Pen 40*(4)	$-1.99x10^{-5}$	(26±11)	-2.06	-1.03	-0.00
Agg 4	0.43	(24±4)	+8.60	+10.32	+12.04
% LMS	-0.15	(22±3)	-3.75	-3.30	-2.85
Agg 80*	-0.29	(10±3)	-3.77	-2.9	-2.03
			-10.52	-0.21	10.08

(1) based on mean - 1 standard deviation
(2) based on mean
(3) based on mean + 1 standard deviation
(4) x commercial traffic factor
*original pavement

Table 13. Calculated Range of Rutting Performance Based on Overlay plus OGFC Data Set - Model II

Predictor	Coeff.	Approx. Range	Theoretical Rut Depth, mm Min.(1)	Ave.(2)	Max.(3)
Intercept	-15.15	--	-15.15	-15.15	-15.15
Agg 40	0.50	(19±4)	+7.50	+9.50	+11.50
Pen 40(4)	$5.5x10^{-5}$	(29±8)	+0.43	+3.11	+5.79
Minus 200	-0.54	(7±1)	-4.32	-3.78	-3.24
Agg 4	0.42	(24±4)	+8.40	+10.08	+11.76
% LMS(4)	$-4.4x10^{-5}$	(22±3)	-3.18	-1.83	-0.48
Agg 80*	0.37				

(1) based on mean - 1 standard deviation
(2) based on mean
(3) based on mean + 1 standard deviation
(4) modified by commercial traffic factor
*original pavement

The percentage of fractured aggregate appears to make a significant contribution to rut resistance in sites 23 and 184

using Model I (Table 10). However, the variable does not enter Model II. Moreover, data in Table 12 show a difference of only 1.5 mm of predicted rutting over the range mean ±1 standard deviation (77-95% of aggregate larger than 4M with at least one fractured face). In other words, the percentage of fractured aggregate is important and the highest possible percentage is desirable, but the variable is fairly well controlled in these pavement layers.

Other aggregate variables contribute to both models. Model I seems to indicate that the amounts of aggregate retained on the 4 and 40 M screens should be decreased and the amount retained on the 80 M increased. However, the ranges in the data set do not permit one to define the limits of these changes, nor do they indicate where the slack may safely be taken up. Similar comments could be made about aggregate variables in Model II. Four of the six variables in Model II do derive from the aggregate gradation, however, emphasizing the importance of this characteristic of the mix.

The penetrations at 40 °F of the asphalt recovered from both overlay and original pavement enter Model I. Both are modified by commercial traffic factor. Over the range of values in the data set, pen 40 (overlay) could contribute as much as 5 mm of rutting. Pen 40 in the original pavement has a negative coefficient, but its contribution is very small. Only pen 40 of the overlay enters Model II.

The percentage of LMS does not make a large contribution in Model I ("saving" from 2.8 to 3.7 mm of rutting). However, the range of LMS contents is rather narrow. When modified by commercial traffic factor, as in Model II, the contribution of % LMS is somewhat greater, but still small.

Model for Overlays With and Without OGFC

In this modelling effort, the data for all overlays and their corresponding original pavements were used. The presence or absence of an open graded friction course was used as a potential variable.

Figure 13 contains a plot of rut depth versus age for this data set. Overlays with OGFC are designated 0, those without by +. This is a larger data set than was used for Models I and II. However, although the maximum age is 11 years, the mean age is 3.3 years and 18 pavements are just two years old. Maximum rut depth is 15.7 mm (about 0.6 inch). In terms of confidence in model building, this situation is marginally better than that for Models I and II.

Table 14 contains information about Model III, constructed for this data set.

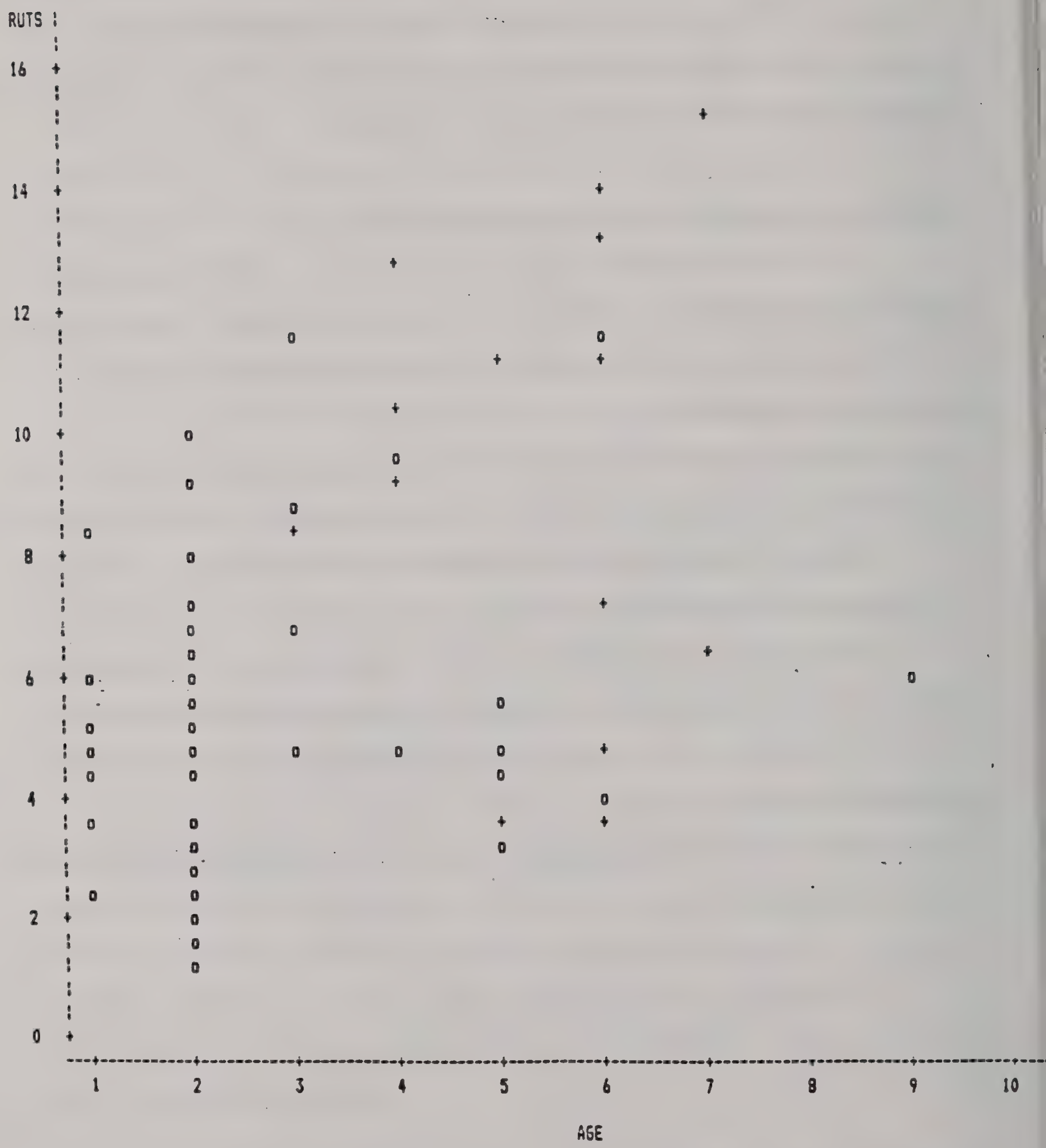


FIGURE 13. Plot of Rut Depth (mm) vs. Age of Pavement for OGFC-containing (+) and other overlays, (0).

Table 14. Model III for all overlay pavements

Variable	Parameter Estimate	Standard Error	Probability
Intercept	15.47	2.57	0.0001
Faces[(1)]	-0.09	0.02	0.0005
OGFC	-2.90	0.62	0.0001
Age*	0.10	0.04	0.0099
Pen. 77	-0.04	0.01	0.0010
Agg. 0.375[(2)]	-0.24	0.09	0.0065
Agg. 80[(3)]	0.39	0.10	0.0004
Pen. 40[(4)]	5.3×10^{-5}	1.1×10^{-5}	0.0001
Agg. 40*[(5)]	6.2×10^{-5}	1.8×10^{-5}	0.0009

F value - 19.82
Adjusted R^2 - 0.6765
Number of observations - 72

*value of variable in original pavement
(1) Percent fractured faces in overlay
(2) Aggregate retained on 3/8 inch (0.375 inch) screen after passing the 0.5 inch screen.
(3) Aggregate retained at 80M after passing the 40M screen.
(4) Penetration at 40°F of asphalt recovered from overlay x commercial traffic factor.
(5) Aggregate retained at 40M for original pavement x commercial traffic factor.

Model III predicts rutting reasonably well as shown by the plot of residuals versus predicted ruts in Figure 14. Rutting in 79 percent of the sites is predicted to within ±2 mm of measured rutting and in 92%, to within ±3 mm.

Of the eight variables in Model III, four are aggregate-related, emphasizing the importance of aggregate gradation and fracture seen in other models.

The mechanism by which age of the original pavement is associated with rutting in the overlay is not obvious. However, the probability is 0.0099 for this variable and it has been

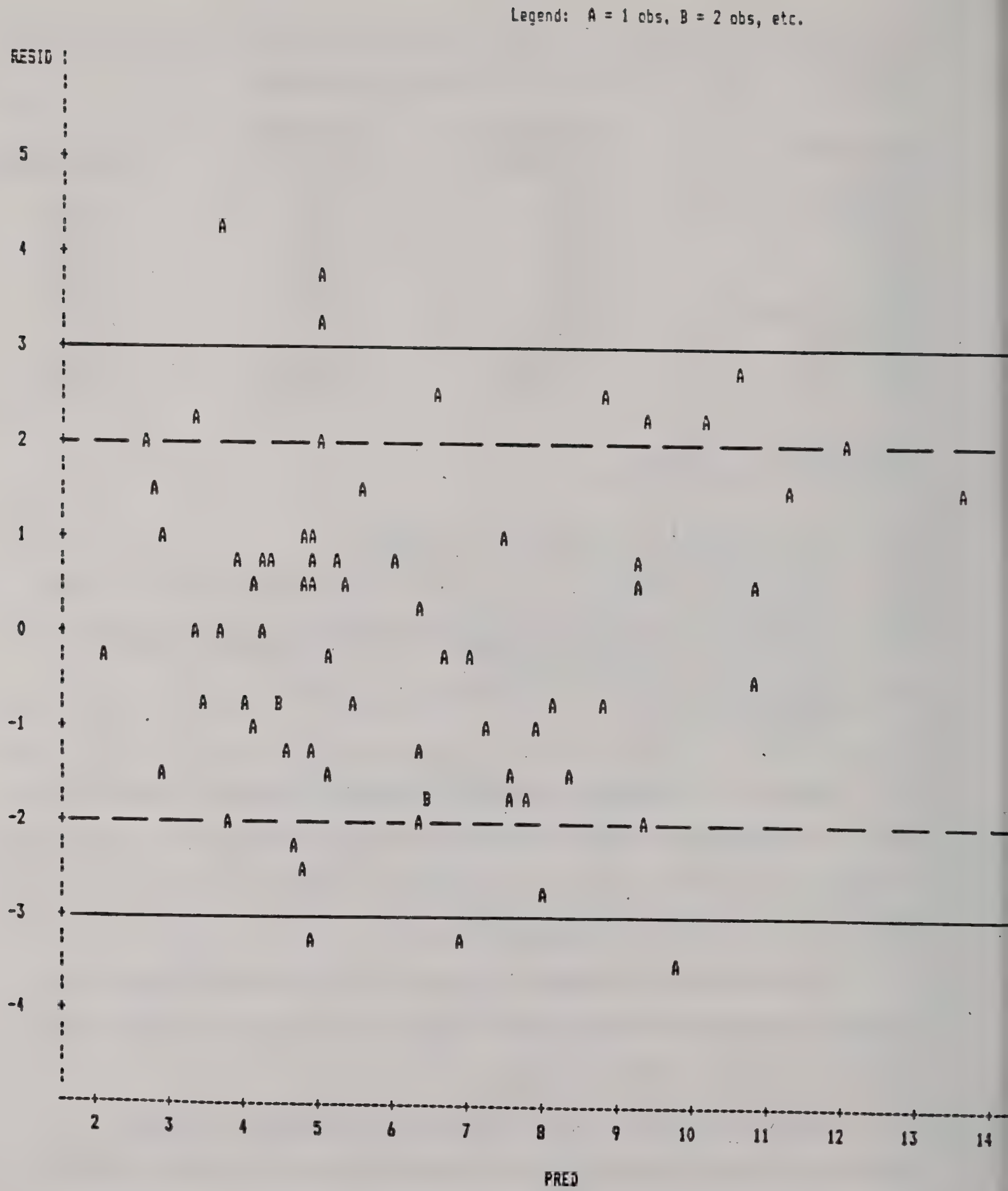


FIGURE 14. Plot of Residuals vs. Predicted Rut Depth (mm) for all overlays.

determined that this age is not a spurious entry resulting from its association with another variable(s). Nevertheless, it is not a controllable quantity.

Calculated potential contributions of each of the variables to rutting based on the mean value ±1 standard deviation for the variables found in the data set are given in Table 15.

Table 15. Calculated range of rutting performance based on data set for overlay pavements, Model III.

Predictor	Coeff.	Approx. Range	Theoretical Rut Depth, mm. Min.[1]	Ave.[2]	Max.[3]
Intercept	15.48	--	15.48	15.48	15.48
Faces	-0.09	85±10	-8.55	-7.65	-6.75
OGFC	-2.90	0 or 1	-2.90	--	0.00
Age*	0.1	--	1.69	1.69	1.69
Pen. 77	-0.04	67±28	-3.80	-2.68	-1.56
Agg. 0.375	-0.24	12.3±2.6	-3.58	-2.95	-2.33
Agg. 80	0.39	9.0±2.5	2.54	3.51	4.49
Pen. 40	5.3×10^{-5}	30.3±8.7	2.85	4.01	5.16
Agg. 40*	6.2×10^{-5}	18.7±4.1	2.86	2.86	2.86
			6.59	14.27	19.04

*for original pavement. Because these variables are uncontrollable at the time an overlay is placed, the mean value of the variable is used in all cases as an example.

The contribution of fractured faces in the aggregate is important, but the differences in that contribution over the range actually found in the data set is rather small. In fact, the same might be said for all of the predictor variables. The "theoretical" range of rutting based on this data set is 0.27 to 0.78 inches. This, no doubt, reflects the nature of the data set. The ranges of values of predictor variables actually found in these overlays are not very wide.

E. Statistical Model for Transverse Cracking

Introduction

Although presently overshadowed by rutting with its immediate safety aspects, cracking continues to be a problem in Montana. HP-GPC studies have indicated that molecular size profile can point toward a tendency to crack. However, even the best asphalts by this criterion sometimes crack. Unfortunately, other contributory factors are not well-defined.

When field performance data were collected for the rutting study, crack counts were made at each sampling site. Therefore, data was available to carry out a statistical modelling study of the relationship of cracking to an extensive list of parameters. This study has been carried out with the help of statistician Tom Kalaris.

The Data

Details concerning the design of the study and the nature of the data are available in previous sections of this report.

As with analysis of the rutting data, the cracking data is complicated by the presence of overlays. In an overlay it is usually not possible to tell whether cracks are reflected from the original pavement or inherent in the overlay itself. Furthermore, little if any cracking data is available for the original pavements. Therefore, it was decided to restrict the analysis to those pavements without overlays. Unless specifically noted, the remainder of this discussion will focus

on those pavements which have not received dense- or open-graded overlays.

Before beginning that discussion, however, it is of interest to look at the condition of the Interstate System with and without overlays in 1986 with regard to cracking. Figure 15 plots the total number of transverse cracks counted in each 500-foot sampling site against the age of the uppermost lift for 137 projects the age of which was known. More than one site may be represented by a point, especially for pavements (mostly overlays) which were two years old. Twenty-three percent of the pavements were in bad condition with regard to cracking. An additional 40% were classified "poor," and 27% "good." Less than 10% were in excellent condition.

There is a general trend toward increased cracks with advancing age, but the scatter of data indicates the influence of other factors. There were a few pavements that remained in good condition after as many as 25 years. However, there were a dismaying number of projects that had deteriorated seriously after only a few years. Particularly disappointing was the performance of a group of overlays constructed in 1984. Of 39 original paving projects overlayed at that time, only six remained in excellent condition after two years; 22 were "good" whereas 18 had already reached the "poor" or "bad" categories.

It was visible in some cases that the cracks in these overlays had reflected from the original pavement. However, it

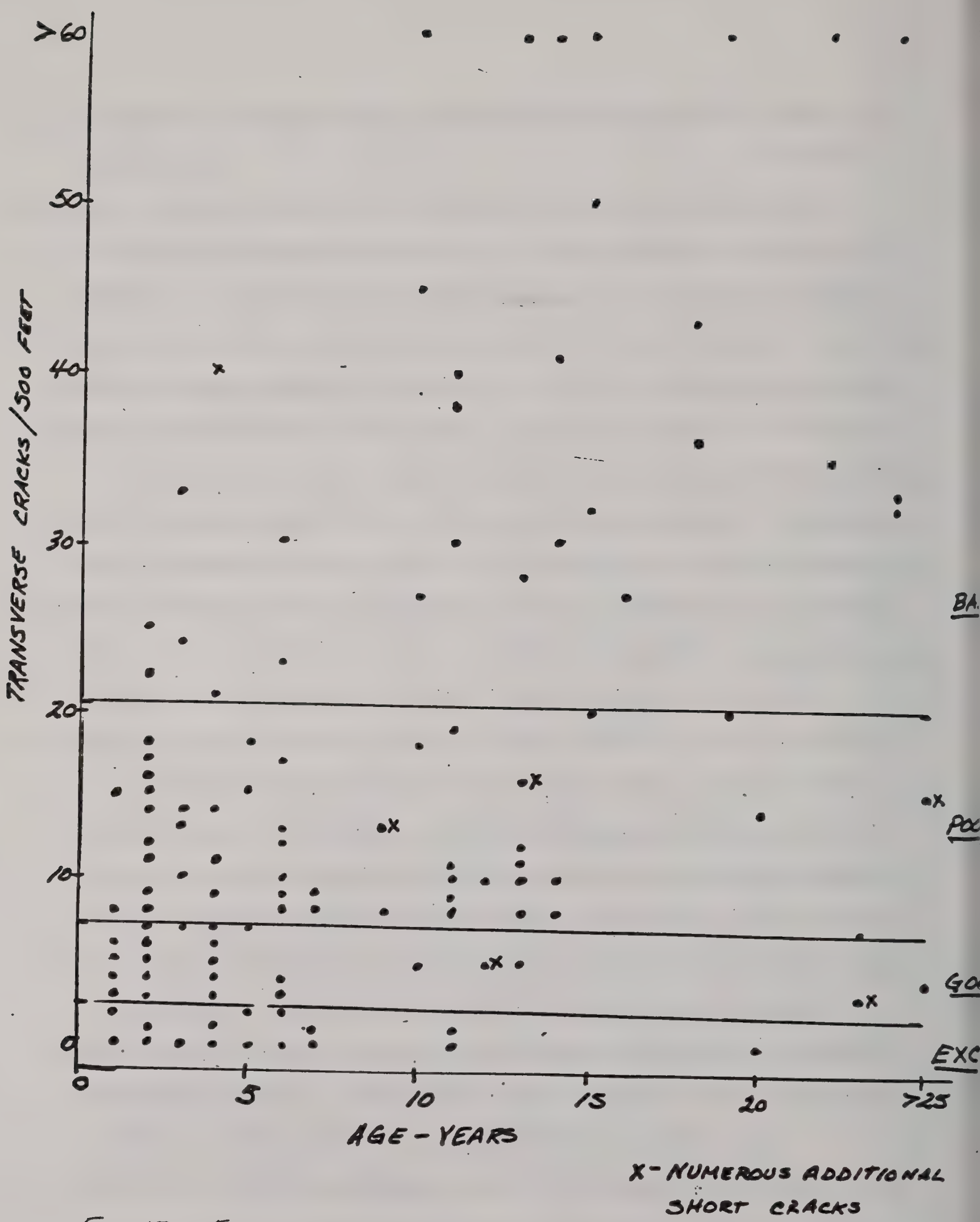


FIG. 15. TOTAL TRANSVERSE CRACKS VS. AGE FOR INTERSTATE PAVEMENTS

was not possible to quantify the extent of reflection. To obtain some basis in fact for a reflective cracking rate, cracks were mapped on a one-mile section of I-90 near Whitehall in June, 1988(MP. 244-245, E.B.). A total of 128 full-width transverse cracks plus some longitudinal and other short cracks were noted. The major transverse and longitudinal cracks had been filled in preparation for the construction of a 0.3' plant mix overlay immediately thereafter [IR 90-5(47)240]. In late March, 1989, the area was resurveyed. A total of 80 reflected full-width transverse cracks were found; no additional transverse or longitudinal cracks were noted. In other words, 65% of the full-width transverse cracks had reflected through in just one winter. While it is true that the winter '88-'89 was a particularly harsh one, it is not possible to ascertain its effect on the cracking rate. [Note: to plot points in Figure 15 for these before-and-after observations, the original pavement was 22 years old and had approximately 12 total transverse cracks/500 feet; the overlay at one year had 8 total cracks/500 feet.]

<u>The Statistical Analysis</u>

Simple correlations of the number of full-width transverse cracks with factors in the lengthy list of possible variables were obtained. No correlations were found between transverse cracking and the following individual variables: penetration grade of original asphalt, asphalt content of pavement, voids content, PVN or other temperature susceptibility measurements, penetrations, %LMS, commercial traffic factor, and Kinematic

viscosity, among others. Values of r^2 for other variables did indicate possible relationships: age ($r^2 = 0.0021$), absolute viscosity ($r^2 = 0.0005$ - this property was skewed by 3 very high values; when these unusual values were omitted, no relationship with cracking was indicated), viscosity temperature susceptibility modified by age ($r^2 = 0.0003$), %SMS modified by age ($r^2 = 0.0003$), PVN modified by age ($r^2 = 0.0001$). However, as will be seen when the model for cracking is discussed, these variables may not contribute to the overall picture.

The Model

The statistical modeling for cracking proved to be very difficult. Numerous efforts to achieve a stable model were made, but results which seemed reasonable on the surface were not viable on statistical grounds. On the other hand, the best model which was achieved, mathematically speaking, is not particularly satisfactory in terms of being helpful to MDOH in its efforts to minimize pavement cracking. Essential information for the model is contained in Table 16.

This model resulted from a square root transformation: the equation for the model is

$$\sqrt{\text{number of cracks}} = a_1 x_1 + a_2 x_2 + \ldots\ldots,$$

where a is the value of the parameter and x is the value of the corresponding variable. Using the data set on which the model was constructed, a plot of the predicted number of cracks against the difference between predicted and actual number can be made.

Table 16. Model for Cracking in Non-Overlayed Pavements

Variable	Parameter[1] Estimate	Standard[2] Error	Probability[3]
Intercept	1.50	2.29	0.5160
% Asphalt[4]	1.01	0.26	0.0002
Age	0.14	0.03	0.0001
Region 4[5]	1.92	0.41	0.0001
Agg.80[6]	-0.14	0.05	0.0141
Agg. 4[7]	-0.13	0.04	0.0036
Minus 200[8]	-0.26	0.11	0.0213
Fvalue	17.363		
Adjusted R^2	0.6091		
No. of observations	63		

(1) The amount by which the value of the variable is multiplied in the equation for the model.

(2) The standard error for the parameter estimate.

(3) An expression of the liklihood that the parameter is actually zero. A value of 0.0500 or less is most desirable (ie., the probability is 5% or less that the parameter value is zero).

(4) Percent of asphalt recovered from roadway cores.

(5) A climate region of the state having highest August temperatures, encompassing much of the eastern portion of Montana.

(6) Percent retained on the 80M screen having passed the 40M.

(7) Percentage retained on the 4M having passed the 10M screen.

(8) Percentage of aggregate which passed the 200M screen.

Analysis of Cracking Data (SQUARE ROOT TRANSFORM (zero values, no Price, no ythick2)) May 15, 1989

Plot of PRESID*PPRED. Legend: A = 1 obs, B = 2 obs, etc.

NOTE: 13 obs had missing values.

FIGURE 16. RESIDUALS VS. PREDICTED VALUES OF THE SQUARE ROOT OF TOTAL CRACKS

This plot (Figure 16) shows that the model predicts cracking moderately well with 94% of the sites being predicted within transverse cracks per 500 feet.

In Table 17 are tabulated the ranges of cracking that would be predicted by the model using the average of each predictor variable actually found in the data set as well as values of the average plus and minus one standard deviation. For this exercise, age has been set at 10 years and climate region is not region 4.

Table 17

Param. x variable	Expected Cracks		
	Max	Ave	Min
Intercept	1.50	1.50	1.50
%Asphalt x 1.01	6.34	5.78	5.22
Age (10) x 0.14	1.40	1.40	1.40
Region 4	0.0	0.0	0.0
Agg 80 x -0.14	-0.97	-1.46	-1.95
Agg 4 x -0.13	-2.53	-3.03	-3.54
Minus 200 x -0.26	-1.32	-1.74	-2.16
cracks =	4.39	2.42	0.44
Expected cracks/500 ft.		20	60.2

There are some problems with the model. First, there is only one asphalt variable represented, that is asphalt content. Furthermore, the parameter is positive, indicating that cracking

increases as asphalt content increases. That might be rationalized in conjunction with another asphalt characteristic, but is more difficult to understand in isolation.

The climate zone variables used in modelling the rutting data also appear in the cracking model. Since those zones were based on highest temperatures, that is surprising. Lowest temperatures would seem to be more reasonably related to cracking. When one examines the average daily temperature data available for the state, one finds that, except for limited areas along the southernmost reaches of I-15, around Butte, and from Terry east on I-94, average temperatures for the three coldest months of the year are very similar. Data on the number of freeze-thaw cycles might be more valuable were it readily available.

Difficulty with the cracking model is disappointing but probably should not be surprising. Caution is always required when attempting statistical analyses on uncontrolled data sets. Crucial variables may be limited in range or may not have been measured, for example.

During the course of these studies, a considerable amount of time has been spent with the data. Although it is not possible to manipulate the large amount of data gathered in this study without the help of a computer, it is possible to find important information within the data set that is not specifically revealed by statistical analysis. The remainder of this section will address these efforts.

Not surprisingly, there is a relationship between the number of cracks and the age of a pavement. An ideal data set would have a full range of pavement ages. In this data set, however, most of the pavements are more than 10 years old and many are nearing the ends of their expected service lives. It is instructive, however, to inspect the data from the aspect of age.

There are just four pavements in this not-overlayed data set that are 5 years old or less. These are 4 years old and three are performing well. Data from the Test Sections can be added, however, and that shows differences in performance which, with the exception of Conoco, can be broadly related to HP-GPC chromatogram with some perturbations from additives. Note in Figure 17 that somewhat better performances (compared to Test Sections) are achieved in two pavements by Conoco at slightly higher LMS percentages. Cenex (without the SMS material seen in the Test Sections and, therefore, with more realistic LMS values) is performing nicely at four years.

In Figure 18 are nine pavement sections 6-10 years old. Only two of these do not have plant mix bases (PMB) which might be expected to slow cracking rates. This slowing of cracking rate may be happening with the Conoco section on PMB. That base was constructed with 200-300 grade asphalt. Conoco with fly ash (10 years) might be envisioned as foreshadowing the performance of the related Test Section. The seven year old Cenex section with base (no SMS shoulder) is not cracking. In the two Phillips

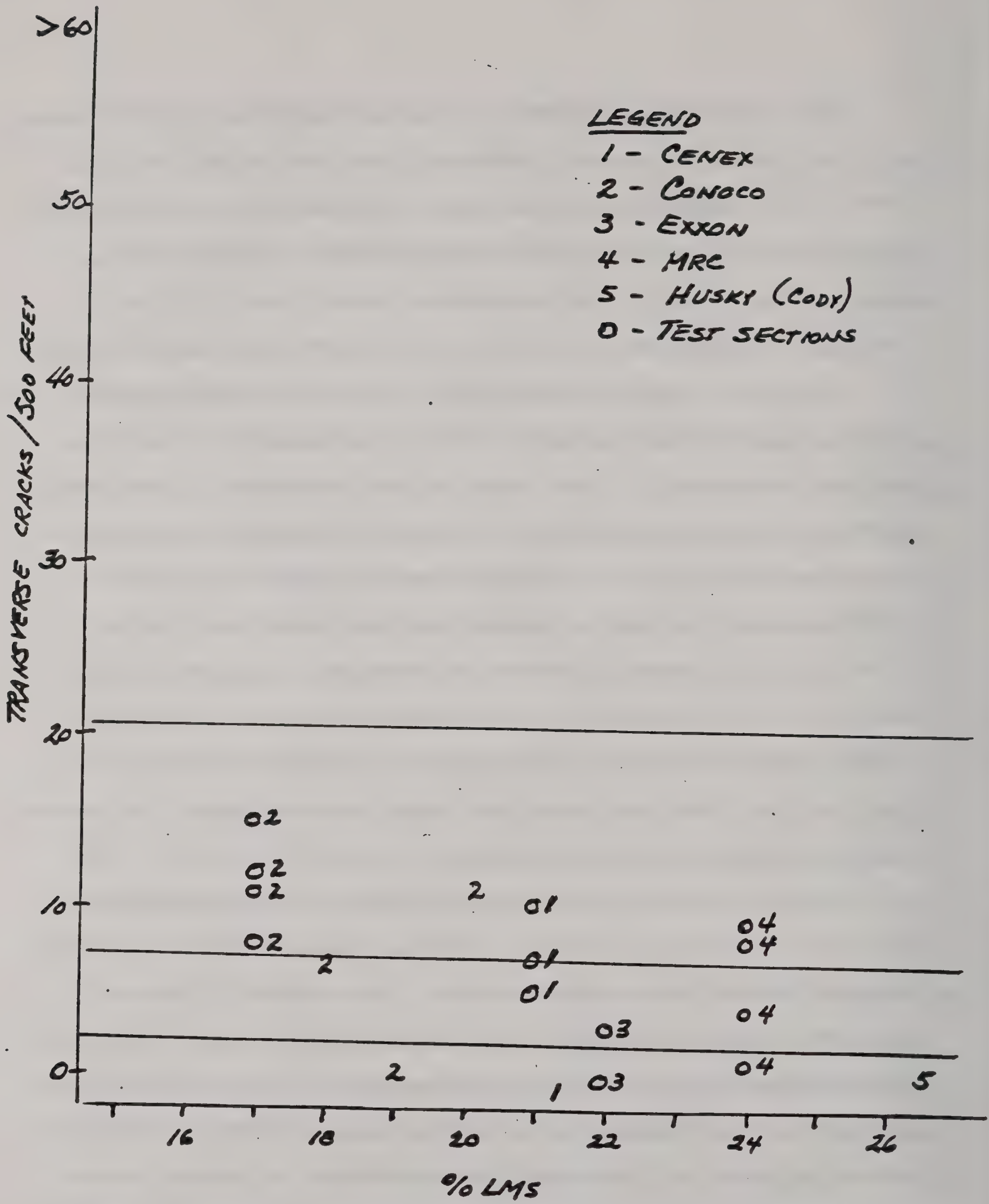


FIG. 17. TOTAL CRACKS IN PAVEMENTS (NOT OVERLAYED), 1-5 YEARS

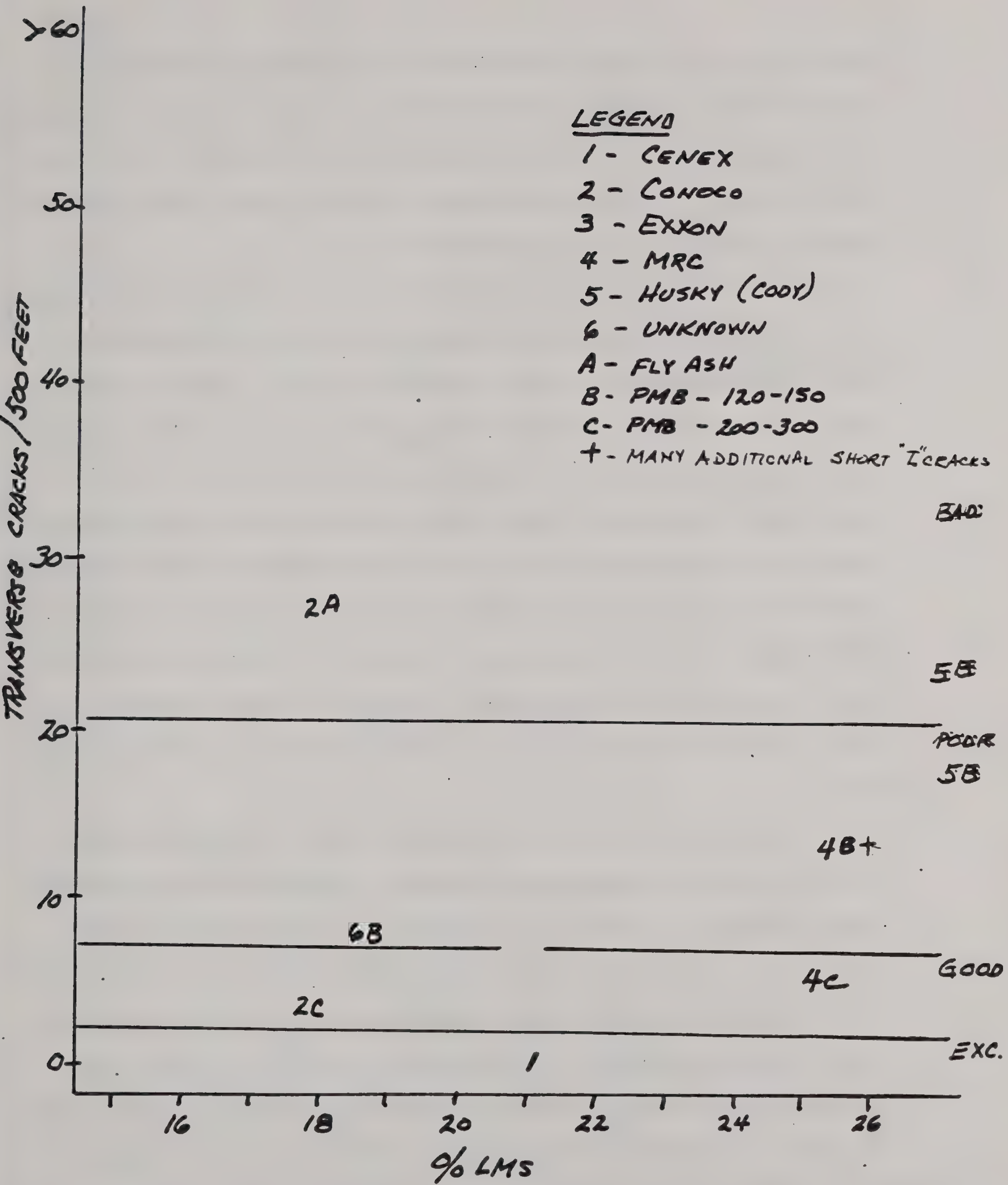


FIG. 18. TOTAL CRACKS IN NON-OVERLAY PAVEMENTS, 6-10 YEARS

sections, 9 & 10 years old, both with slightly higher LMS contents than comparable Test Sections, that with a 200-300 grade PMB is performing much better than the section with lime and a 120-150 base. Two Husky sections, both six years old, at more than 27% LMS, are cracking severely.

Most of the pertinent pavements are between 11 and 15 years old. One might expect the broadest range of performance here and, indeed, Figure 19 shows that to be true. There are only four pavements with LMS percentages above 20; all of them are in poor condition. Only four pavements remain in the "good" to "excellent" categories; their LMS contents range approximately from 16 to 19%. Most of the rest of the pavements with less than 19% LMS are in the "poor" class, albeit in the better portion of that range. Many of the mixes in Figure 19 contain fly ash or lime, some have plant mix bases. However, none of these, or of other factors available, show a relationship to cracking in this data set.

All of the asphalts in pavements under 15 years old were 120-150 grade at construction (except two 200-300 pen asphalts used in PMB's). However, in pavements 16 years or more old (Figure 20), two of 14 used 85-100 pen asphalt, seven used 100-120, four used 150-200; only one pavement was constructed with 120-150 grade material. The best and the worst of these old sections were paved with 100-120 asphalt. That with about 20% LMS is in excellent condition. However, most of the asphalts with less than 20% LMS are in bad condition. Two other pavements

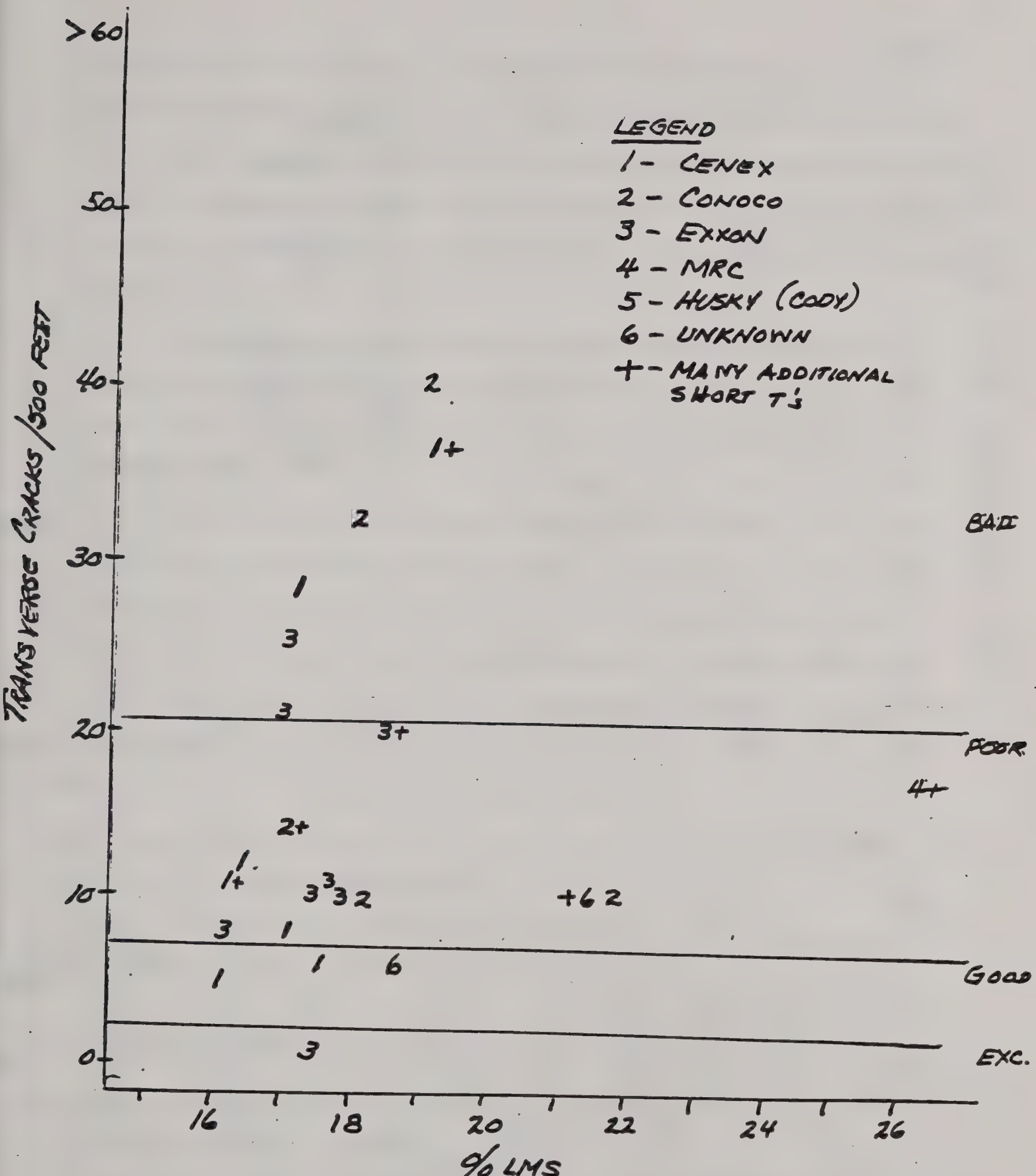


FIG. 19. TOTAL CRACKS IN NON-OVERLAY PAVEMENTS, 11-15 YRS.

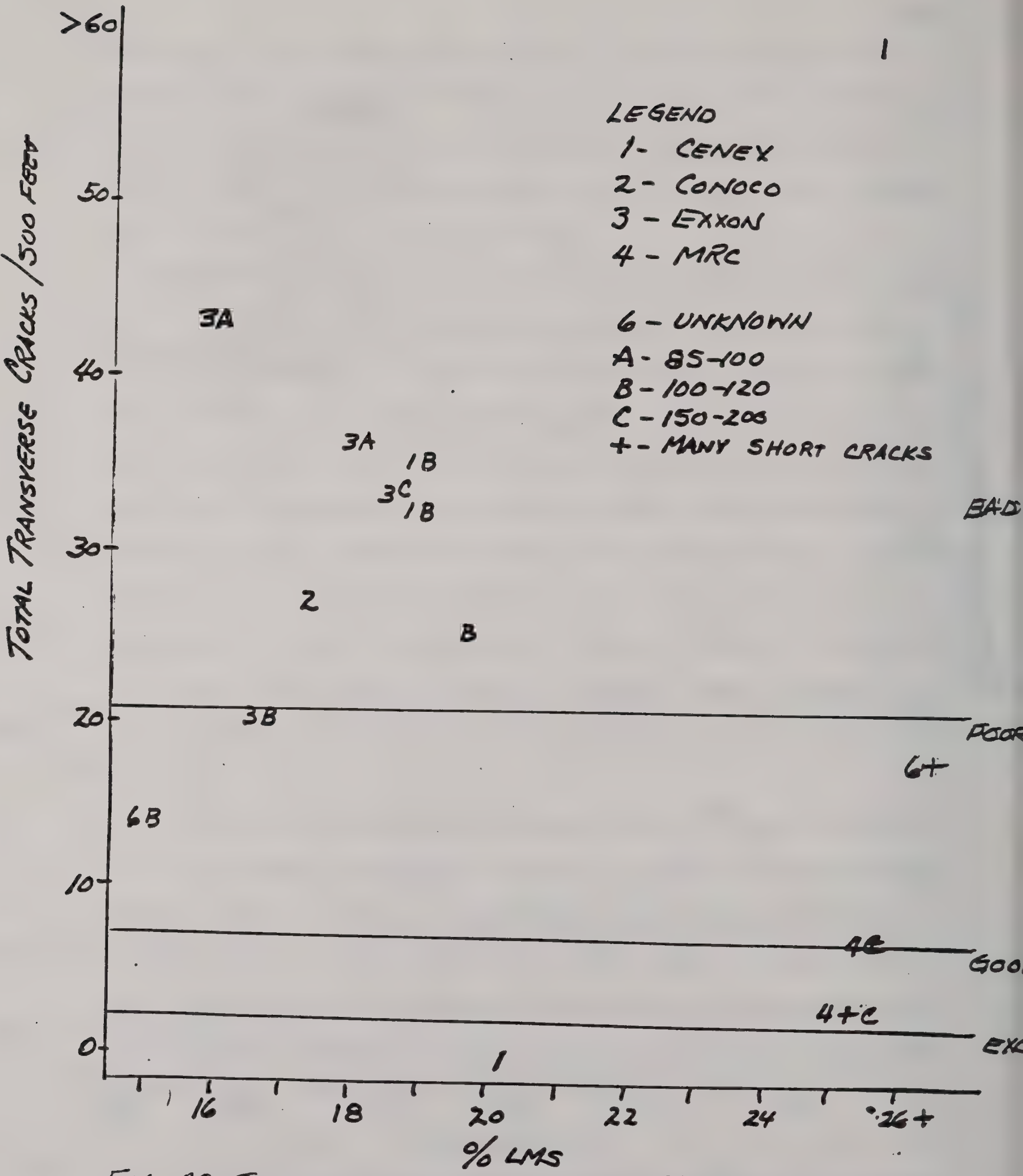


FIG. 20. TOTAL CRACKS IN NON-OVERLAY PAVEMENTS, ≥ 16 YEARS.

in good condition after 23 years were constructed with 150-200 pen grade asphalt with about 25% LMS. It is interesting to note that the asphalts on which the Montana model was based were also 150-200 grade. It should also be noted that the rutting in these two very old sections was less than 0.5 inch in spite of the soft original asphalt.

Because of the differences in performance among products from different refineries indicated by the Test Sections, the data from non-overlayed pavements were also studied refinery by refinery.

Twelve sections containing Cenex asphalt range in age from 4 to 24 years, averaging about 15 years. Two are in excellent condition, one of these is 20 years old, the other only four years old. Two more are in good condition, the remainder are "poor" and "bad." There are no perceptible patterns in the group as a whole.

Table 18. Cenex Sections on I-15

Total Cracks	Additive	%AC	PMB	%LMS	%Voids	Vis	PVN	Pen Ratio
5	Lime	6.1		16.1	6.0	1258	-0.98	0.55
6	Fly	5.8		17.6	3.4	1982	-0.97	0.43
8	Flyash	6.2	X	17.6	4.9	1523	-1.03	0.48
11+	Lime	6.9	X	16.3	2.2	1240	-0.99	0.45
12	Lime	6.2	X	16.5	2.8	1036	-0.97	0.48
36+	Lime	5.4		19.3	5.3	2810	-0.81	0.71

+Numerous Short Transverse

There is, however, a set of six pavements, all constructed in 1973 between mile posts 39 and 94 on I-15 using 120-150 grade Cenex asphalt. These sites constitute inadvertent "test sections" although the data set is too small for dependable statistical analysis (see Table 18). Two of the pavements are in good condition, three are "poor," one "bad." Without belaboring the details, it may be said that there are no obvious trends in the set. However, that project which has the most severe cracking also has the highest LMS content, the highest viscosity and the lowest asphalt content of the six pavements, as well as the lowest PVN and the highest pen ratio. If the historical data is accurate and this section was paved with the same material at roughly the same time, it is not possible to explain these discrepancies in the character of the aged asphalt with the known data. It might be speculated that the mix was overheated contributing to increased LMS content and excessive viscosity; perhaps the aggregate used was more reactive.

Eleven sections have been paved with Conoco products. The sections range in age from 4 to 16 years (10 years average) and the performance from excellent to bad. None of the sections more than 10 years old is in good condition. Five of the seven older sections are in the parts of the state with coldest winter temperatures but no other patterns are obvious.

Among the four younger sections (4, 5 and 6 years), that with the worst cracking performance has the highest LMS (still only about 20%, however) and the highest absolute viscosity of

any of the Conoco pavements, regardless of age. Reasons for this are unknown but one might speculate, as for the Cenex situation discussed earlier, about higher mix temperatures and/or more reactive aggregate.

Pavements with Exxon asphalts range in age from 10 to 24 years. Of the 14 projects, only two remain in good condition, seven are "poor." Fly ash was used in six projects, four of which are among the better performers of the group. (In the Test Sections, the Exxon trial with the most cracking is that with fly ash.) Limestone dust was used in four sections 18 and 19 years old, all are in bad condition.

Five adjacent sections, 10, 11 and 12 years old, form a possible "test section." The performance ranges from excellent to bad. The only apparent trends within the data are that PVN and penetration ratios differ significantly for the more seriously cracked sections (data for recovered asphalts). (Table 19). In fact, the cores from the best of those pavements were seriously stripped upon arrival in this laboratory. None of the others exhibited stripping.

Table 19. Five Sections with Exxon Asphalt

Total Cracks	Age	Additive	%AC	PMB	%LMS*	%Voids	Vis	PVN*	Pen* Ratio
1	11	Flyash	5.6		17.5	2.7	3396	-0.66	0.57
10	12	Flyash	5.2	X	17.8	1.4	8128	-0.80	0.50
11	11	--	4.5	X	17.7	6.9	9507	-0.65	0.56
18	10	--	5.2		17.9	1.4	6006	-0.97	0.48
25	12	--	5.1		17.2	5.1	2210	-1.08	0.49

*recovered AC

Pavement constructed with asphalts from the refinery at Great Falls (now Montana Refining Company, previously Simmons, nee Phillips) were among the more interesting of this study. The data in Table 20 shows three pavements, ages 9, 11 and 13, in poor condition. These are not surprising, based on the experience of the Test Sections. There is a 10 year old pavement in the "good" category, which would not be expected. However, it is on a plant mix base of 200-300 grade A.C. Even more surprising are the two good 23-year old sections. These were both constructed with 150-200 grade asphalt.

One might be tempted to say that pavement should be constructed with 150-200 grade material, or with 100-120 using mineral filler to avoid cracking. Obviously, there are contradictions to such assumptions. And in the end, the data from the Interstate survey leaves us unable to draw firm conclusions about the causes of transverse cracking at least in part because we are using uncontrolled data. We do have,

fortunately, the Test Sections which continue to show obvious differences in performance in a reasonably well-controlled experiment.

Table 20. Sections Containing MRC Asphalt

Total Cracks	Age	Additive	%AC	PMB	%LMS*	Grade	Vis	PVN*	Pen* Ratio
3+	23	?	5.5		24.8	150-200	2766	-0.81	0.53
5	10		5.3	X(1)	25.2	120-150	1245	-0.81	0.46
7	23	?	5.5		25.3	150-200	7808	-0.50	0.71
13+	9		5.2	X	25.4	120-150	5468	-0.64	0.40
16+	14		6.6	X	26.3	120-150	1536	-0.47	0.32
19	11	Flyash	5.7		28.5	120-150	6158	-0.82	0.59

+ many additional short transverse cracks

(1) PMB constructed with 200-300 grade AC

IV. IMPLEMENTATION

Some of the results of the work described in this report can be implemented directly. Other results point to the need for further work.

Aggregate variables

The method used for determining the contribution of aggregate gradation to rutting is not entirely satisfactory and should be investigated further. Nevertheless, the influence of two aggregate sizes is strongly indicated.

The model indicates that an increase in the percent of aggregate retained on the 10M screen coincides with a decrease in rutting. Obviously, that can be carried to extremes but it is worth noting that the range in values for this variable in the sample set is 7.1% to 26.2%. The anti-rutting specifications require 12 to 24% ± 6% job mix tolerance. This permits a wider range than is actually found in the non-overlayed Interstate pavements. However, since the aggregate retained on the 10M screen can apparently "save" or help to prevent a maximum of perhaps 9 mm and a minimum of 6 mm of rutting over the life of the pavement, it may be worth the effort to tighten up this requirement somewhat.

The apparent effect of higher percentages of minus 200M aggregate is almost directly opposed to the effect described above. That is, in the range of 5.0 to 8.0%, minus 200 aggregate can contribute to 5 to 8 mm of rutting. The range of minus 200 percentages in the data set is 3.8 to 11.6; the range permitted

by the anti-rutting specification is 5 to 7% ± 2%. This research points to the desirability of decreasing that range somewhat.

According to the model, about 90% fractured aggregate can help to eliminate up to 15 mm of rutting in the 15 year life of a pavement, i.e., about 1 mm per year. The anti-rutting specification requires at least 70% of the coarse aggregate to have at least one fractured face. The feasibility of raising that requirement should be investigated.

Asphalt Cement Variables

The range of asphalt cement contents in the data set is 4.5 to 7.2 percent. Nevertheless, the statistical analysis shows that asphalt content has a relationship to the depth of rutting in that increased asphalt content parallels increased rutting.

It seems that asphalt content has been difficult to control, for, although design asphalt content correlates with core asphalt content (r^2=0.533, P=0.0001), inspection of individual projects shows enough variation to make one wince. It is recommended that asphalt content be kept as low as possible within limits of good design. Moreover, it is important to find a means of controlling asphalt content continuously during the paving operation and to control adherence to design requirements more closely.

The relationship of the consistency measurements penetration and viscosity to rutting will require some additional effort to implement. The change in these parameters with time in Montana pavements is not known, but the Test Sections offer an opportunity to gain that knowledge. Also, the interrelationship

among penetration and kinematic and absolute viscosities is quite complex and evidently is not adequately described by present temperature susceptibility parameters.

Voids

The model shows a trend of increased rutting with increased voids. In fact, a value of about 2.4% voids apparently would contribute only 0.5 mm of rutting over the first six years of pavement life. However, 8.6% voids would contribute only 4.2 mm in 15 years and 13% voids might contribute to about 7 mm of rutting in 15 years. Therefore, although voids control may be important for a variety of reasons, by itself, high voids content does not seem to be a major source of rutting in Montana Interstates. It does not follow that 0% voids would result in no rutting. The fact that the model does not indicate increased rutting at extremely low voids content may be only a reflection of lack of data for pavements of that type.

It seems wise, therefore, to keep voids content at a low, but not extremely low, level.

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Apppendix A Pilot Study Results

In this appendix, the data from the various tests conducted in the Rutting Pilot Study are summarized. For each parameter the mean value as well as the range (minimum to maximum value) are given for each project in the driving lane (D) or shoulder (S). In addition the data for driving lane and shoulder are combined for each project (labeled "both"). Next, the data for all projects in the driving lane or and all projects in the shoulder are compiled. Finally, figures are given which indicate the level of confidence at which we can believe that the differences seen do not result from chance. The smaller the P value, the more likely it is that the differences do not result from chance.

Appendix A.1 Historical Data

	Proj. 1	Proj. 2	Proj. 3
Location	I 94, mp. 46.7	I 90, mp. 182.6	I 90, mp. 196
Year const.	1974	1974	1978
Asphalt, Design:			
Refiner, grade	"C"/120-150	"A"/120-150	"B"/120-150
per cent	5.6	6.3	6.6
Marshall Stab/ flow	1096/8	1422/10	1100/9
Density	2.36	2.39	2.32
Voids %	4.7	3.4	3.9
Rut depth, in.	>0.75	0.5-0.75	insignificant

Appendix A.2 Marshall Stability

Project	Lane	Mean	Min. Max.
1	D	1892	1605 - 2179
	S	1823	1690 - 1957
2	D	2085	1607 - 3262
	S	1536	1311 - 1688
3	D	1817	982 - 2526
	S	1571	1242 - 1872
1	Both	1858	1605 - 2179
2	Both	1810	1311 - 5261
3	Both	1694	982 - 2526
All	D	1919	982 - 3261
	S	1601	1242 - 1957

	P-values
Lane(1)	0.1385
Proj. x Lane(2)	0.5643
Proj.(3)	0.6010

Appendix A.3 Marshall Flow

Project	Lane	Mean	Min. Max.
1	D	5.5	5.0 - 6.0
	S	6.0	4.7 - 5.2
2	D	6.4	5.7 - 8.1
	S	7.0	6.5 - 7.7
3	D	6.8	5.2 - 8.5
	S	8.0	7.1 - 8.5
1	Both	5.2	4.7 - 6.0
2	Both	6.7	5.7 - 8.1
3	Both	7.4	5.2 - 8.5
All	D	5.0	5.0 - 8.5
	S	4.7	4.7 - 8.5

	P-values
Lane(1)	0.0101
Proj. x Lane(2)	0.0248
Proj.(3)	0.0142

(1) Significance of differences between driving lane and shoulder, same project.
(2) Significance of differences between projects in either driving lane or shoulder.
(3) Significance of differences between projects, all data.

Appendix A.4 % Asphalt (Direct)

Project	Lane	Mean	Min. - Max.
1	D	5.1	5.0 - 5.2
	S	5.0	4.9 - 5.1
2	D	5.7	5.3 - 6.1
	S	5.4	5.1 - 5.7
3	D	5.7	5.3 - 6.1
	S	5.8	5.6 - 6.0
1	Both	5.0	4.9 - 5.2
2	Both	5.6	5.1 - 6.1
3	Both	5.8	5.3 - 6.1
All	D	5.6	5.0 - 6.1
	S	5.5	4.9 - 6.0

	P-values
Lane	0.0704
Proj. x Lane	0.0717
Proj.	0.0364

Appendix A.5 % Asphalt (by difference)

Project	Lane	Mean	Min. - Max.
1	D	5.92	5.9 - 5.94
	S	5.55	5.5 - 5.6
2	D	7.0	5.9 - 8.1
	S	6.9	6.2 - 8.4
3	D	6.6	6.4 - 6.8
	S	6.2	5.8 - 6.5
1	Both	5.7	5.5 - 5.9
2	Both	7.0	5.9 - 8.4
3	Both	6.4	5.8 - 6.8
All	D	6.6	5.9 - 8.1
	S	6.3	5.5 - 8.4

	P-values
Lane	0.5329
Proj. x Lane	0.9021
Proj.	0.0011

Appendix A.6 Penetration - 40°F

Project	Lane	Mean	Min. -Max.
1	D	46.5	36.0 - 57.0
	S	39.7	31.9 - 47.3
2	D	38.8	35.3 - 44.0
	S	26.7	25.0 - 29.0
3	D	28.8	24.0 - 43.0
	S	22.0	19.0 - 30.0
1	Both	43.1	31.9 - 57.0
2	Both	31.9	25.0 - 44.0
3	Both	25.4	19.0 - 43.0
All	D	34.8	24.0 - 57.0
All	S	26.6	19.0 - 47.5

	P-values
Lane	0.0592
Proj. x Lane	0.6994
Proj.	0.0021

Appendix A.7 Penetration - 77°F

Project	Lane	Mean	Min.- Max.
1	D	89.2	76.3 - 102.0
	S	76.8	66.1 - 87.5
2	D	87.8	66.3 - 107.0
	S	60.5	45.0 - 85.0
3	D	63.9	48.0 - 101.3
	S	43.4	32.3 - 59.0
1	Both	83.0	66.1 - 102.0
2	Both	72.2	45.0 - 107.0
3	Both	53.6	32.3 - 101.3
All	D	75.0	48.0 - 107.0
	S	54.7	32.3 - 87.5

	P-values
Lane	0.0552
Proj. x Lane	0.0622
Proj.	0.0064

Appendix A.8 Penetration - 90°F

Project	Lane	Mean	Min. -Max.
1	D	156.3	145.0 - 163.0
	S	141.2	118.5 - 164.0
2	D	165.2	149.0 - 197.0
	S	103.0	92.0 - 118.0
3	D	103.5	83.0 - 119.0
	S	84.1	62.3 - 122.0
1	Both	146.3	118.5 - 164.0
2	Both	129.9	92.0 - 197.0
3	Both	93.8	62.3 - 122.0
All	D	127.3	83.0 - 197.0
	S	100.0	62.3 - 164.0

	P-values
Lane	0.0070
Proj. x Lane	0.1016
Proj.	0.0209

Appendix A.9 Viscosity - 140°F

Project	Lane	Mean	Min. - Max.
1	D	1030	140 - 1920
	S	1538	1111 - 1965
2	D	2294	1646 - 3219
	S	2582	2200 - 2971
3	D	2245	535 - 3286
	S	2485	1645 - 3149
1	Both	1284	140 - 1965
2	Both	2438	1646 - 3219
3	Both	2365	535 - 3286
All	D	2058	140 - 3286
	S	2360	1111 - 3149

	P-values
Lane	0.3052
Proj. x Lane	0.9235
Proj.	0.0235

Appendix A.10 Viscosity at 275°F

Project	Lane	Mean	Min. - Max.
1	D	270	259 - 281
	S	320	281 - 358
2	D	271	248 - 280
	S	370	327 - 417
3	D	367	318 - 436
	S	407	308 - 455
1	Both	295	259 - 358
2	Both	320	248 - 417
3	Both	387	308 - 455
All	D	319	248 - 436
	S	380	281 - 455

	P-values
Lane	0.0136
Proj. x Lane	0.2285
Proj.	0.0736

Appendix A.11 Ductility - 60°F

Project	Lane	Mean	Min. - Max.
1	D	38	23 - 53
	S	38	36 - 41
2	D	86	82 - 90
	S	26	22 - 30
3	D	34	15 - 53
	S	24	7 - 70
1	Both	38	23 - 55
2	Both	56	22 - 90
3	Both	29	7 - 70
All	D	45	15 - 90
	S	27	7 - 70

	P-values
Lane	0.0014
Proj. x Lane	0.0032
Proj.	0.2257

Appendix A.12 Bulk Specific Gravity

Project	Lane	Mean	Min. - Max.
1	D	2.42	2.42 - 2.42
	S	2.41	2.39 - 2.42
2	D	2.40	2.38 - 2.43
	S	2.34	2.31 - 2.37
3	D	2.33	2.37 - 2.37
	S	2.28	2.23 - 2.33
1	Both	2.41	2.39 - 2.42
2	Both	2.37	2.31 - 2.43
3	Both	2.30	2.23 - 2.36
All	D	2.37	2.31 - 2.43
	S	2.32	2.23 - 2.42

	P-values
Lane	0.0582
Proj. x Lane	0.6164
Proj.	0.0086

Appendix A.13 Rice Specific Gravity

Project	Lane	Mean	Min. - Max.
1	D	2.48	2.47 - 2.50
	S	2.49	2.48 - 2.50
2	D	2.40	2.38 - 2.43
	S	2.44	2.42 - 2.48
3	D	2.43	2.40 - 2.47
	S	2.43	2.40 - 2.48
1	Both	2.49	2.47 - 2.50
2	Both	2.44	2.42 - 2.51
3	Both	2.43	2.40 - 2.48
All	D	2.45	2.40 - 2.51
	S	2.44	2.40 - 2.50

	P-values
Lane	0.8658
Proj. x Lane	0.7092
Proj.	0.3028

Appendix A.14 % Voids

Project	Lane	Mean	Min. - Max.
1	D	2.41	2.14 - 2.68
	S	3.22	3.10 - 3.34
2	D	2.33	
	S	2.44	2.42 - 2.48
3	D	4.01	2.11 - 6.51
	S	6.59	3.46 - 9.83
1	Both	2.82	2.14 - 3.34
2	Both	2.44	2.42 - 2.51
3	Both	5.30	2.11 - 9.83
All	D	3.18	
	S	1.30	1.30 - 9.83

	P-values
Lane	0.0211
Proj. x Lane	0.4266
Proj.	0.4974

Appendix B. Experimental

Bulk Specific Gravity - MT-314, Method B (AASHTO T 166-78)

Rice Specific Gravity - MT-321 (Modified AASHTO T 209-82)

Marshall Stability & Flow - MT-306 (AASHTO T 245-82)

Aggregate Gradation - Modified[1] MT-202/MT-307 (AASHTO T 27-82)

Fractured Faces - Modified[2] MT-217

Penetration - AASHTO T 49-80

Viscosity

Kinematic - AASHTO T 201-80

Vacuum Capillary - AASHTO T 202-80

Ductility - AASHTO T 51-81

Percent Asphalt - MT-307; Modified[3] AASHTO T 170-82

1. Sieve sizes used: 1½", 1", 3/4", ½", 3/8", #4, #10, #40, #80, #200. Due to the large quantity of samples and time required, washing the aggregate was eliminated. Therefore, shaking time was increased from 5 to 15 minutes.
2. To distinguish fractured faces from coring and lift cutting, mechanically cut faces were colored using a red indelible marker while core samples were still intact. Only those fractured faces without the red colored surface were counted. (Markers by Penn Manufacturing Corporation, Passaic, N.J., type X90 Red will withstand 60°C water immersion and trichloroethylene solvent extraction.)
3. AASHTO T 170 specifies centrifuging the trichloroethylene/asphalt solution in wide-mouth bottles to

remove fines prior to concentrating by distillation. This procedure has been modified as follows:

> The solution from the extraction is filtered through tared 12.5 cm Whatman GF/A glass microfibre filters using a 120 mm Buchner funnel, 2.0 liter vacuum flask, and vacuum pump. Filter papers are changed as flow rate necessitates and rinsed with solvent thoroughly prior to change. The filtration is repeated and papers are set aside for quantitation in percent asphalt/aggregate recoveries.

SUGGESTED METHOD FOR DETERMINATION OF THE MOLECULAR SIZE DISTRIBUTION OF ASPHALT CEMENT BY HIGH PERFORMANCE GEL PERMEATION CHROMATOGRAPHY

Scope

This document covers the determination of molecular size distribution of asphalt cement by High Performance Gel Permeation Chromatography (HP-GPC). It involves: 2) a method for extraction of asphalt from roadway mixes using tetahydrofuran (THF) as solvent, 3) a method for preparing the asphalt for HP-GPC analysis and 4) a method for the HP-GPC analysis.

A. METHOD FOR EXTRACTION OF ASPHALT CEMENT FROM MIXTURES WITH AGGREGATE

Equipment and Materials

-core saw

-vise, or alternative, to crush core

-1000 ml Teflon beakers

-500 ml Büchner funnel

-glass fibre filters (eg. Whatman GFIA)

-ultrasonicator (eg., an instrument suitable for biological use in the disruption of cell membranes, similar to Bronwill "Biosonik" Brownstone BP-1)

-tetrahydrofuran, HPLC grade

-filtering flask

-tubing to connect to aspirator

-rotary evaporator with water and oil baths

-500 ml round bottom glass flasks with standard taper, ground glass joint suitable for the rotary evaporator

-5-10 dram glass vials with polyethylene or Teflon seals

Procedure

1. Using a saw, remove the uppermost 0.5" of the core as well as materials from the interfaces between lifts as these portions may contain material not representative of the bulk of the asphalt cement.
2. Lightly crush the sample (a vise works well) and place up to 300 g in a Teflon beaker
3. Cover the sample with 300-500 ml THF. (Handle THF in fume hood.)
4. Sonicate the sample (at about 40% of output, 0-120 watts) for 40 minutes.
5. Filter by aspiration through a glass fibre filter in the Büchner funnel. Thoroughly wash the aggregate with additional THF. The sonification step may be repeated, if necessary, using fresh solvent.
6. Refilter the asphalt solution to ensure removal of fine sediment.
7. Transfer the solution in portions to a 500 ml round bottom flask and roto-evaporate using a

water bath at 40°C and aspiration to remove most of the solvent.

8. Replace the water bath with an oil bath at 90-100°C and continue solvent removal for an additional 40 minutes.

9. Pour the recovered asphalt into a glass vial, seal and store.

B. PREPARATION OF ASPHALT CEMENT FOR HP-GPC ANALYSIS.

This portion of the procedure describes the preparation of asphalt cement, as received from the refiner or as extracted from a mix, for analysis by HP-GPC.

Equipment and Material

- -Analytical balance
- -5-10 dram glass vials with Teflon or polyethylene closures
- -12 ml glass centrifuge tubes
- -25 ml buret
- -bench-top centrifuge
- -tetrahydrofuran (THF), HPLC grade, dry (<0.05 % water), drawn from the solvent reservoir of the HP-GPC instrument

Procedure

1) Accurately weigh a small amount of asphalt cement (generally 0.0200 to 0.0500 g) into a glass vial.
2) By means of the buret, add sufficient THF to prepare a 0.5% (w/v) solution. Cap and shake to dissolve asphalt.
3) Transfer the solution to a centrifuge tube and centrifuge for 10 min.
4) Decant solution for use in HP-GPC analysis.

Notes: Prepare sample just prior to use for most consistent results.

The centrifugation step is particularly important when using asphalts extracted by ultrasonification. They may contain extremely fine particulate matter that can damage HP-GPC columns.

C. DETERMINATION OF MOLECULAR SIZE DISTRIBUTION OF ASPHALT CEMENTS BY HIGH PERFORMANCE GEL PERMEATION CHROMATOGRAPHY.

Equipment and Materials

-HPLC instrument consisting of the following elements

a) pump capable of delivering 0.1 to 9.9 ml/min.

b) injector

c) pre-column filter.

d) HP-GPC column set: 1-1000 Å and 2-5000 Å ("Ultrastyragel" Waters, Inc.*), in series.

e) tubing and fittings compatible with instrument to make shortest possible connections between elements of the instrument.

f) Detector - absorbance detector. 340 nm*
- differential refractometer

g) Flow meter, in-line, digital*

-syringe and needle - 250 ul for use with loop injector
- 150 ul for use with non-loop injector

-solvent reservoir - 4 liter bottle fitted with a drying tube containing $CaSO_4$ (Drierite) to allow pressure equalization; a tube connected to a source of helium; a tube leading to the pump inlet valve.

-temperature control device to maintain 26°C for solvent, columns and RI detector

-data recording device

-tetrahydrofuran, HPLC grade, dry (less than 0.05% H_2O)

-pyridine

-suction filter apparatus

-0.45 u silver metal membrane filters* or 0.45 u filter compatible with THF

-waste solvent reservoir

*Recommended

Procedure

1) Start temperature control system.

2) Handle THF as much as possible in a fume hood.

3) Filter sufficient fresh, dry THF through the 0.45μ filter to fill the solvent reservoir.

4) Maintain a very slow flow of helium gas through the solvent to remove entrained air.

5) Start pump and increase the flow rate slowly (follow manufacturers' instructions to avoid damage to the columns).

6) Bring system to equilibrium. A state of equilibrium is attained when flow rate remains stable, temperature is stable and baseline of refractometer is flat. Finally, successive injections of the same asphalt sample should be identical. The length of time required will depend on temperature changes and length of time the system has not been used--under ordinary circumstances, about an hour. This time may be shortened by keeping the temperature controlled at all times and by maintaining a constant low flow (0.1 ml/min) of solvent through the system. For the latter purpose, the solvent may be recycled.

7) Prepare asphalt sample as per section B.

8) Inject 100 ul of 0.5% w/v asphalt in THF and simultaneously start data collection system, if that is not done automatically.

9) Monitor flow rate. Fluctuation of ± 0.005 ml/minute may be considered to seriously compromise the data.

General Notes

Extraction (Section A)

We have used the ultrasonification procedure because it is very efficient and amenable to the small quantities needed for the HP-GPC analysis. We have not tried any of the standard extraction methods using THF as a solvent. There is no reason why these should not be effective. However, THF is more expensive than the "normal" solvents. Also, some extractors are not designed to recover the asphalt solution.

We have used a rotary evaporator to remove solvent from asphalt. The principle of the "roto-vap" and the low boiling point of THF (62°C) combine to reduce the exposure of the asphalt to heat.

Use of asphalts which have been extracted with halogenated solvents is not recommended because of chemical reactions which appear to affect the molecular size distribution. It is suggested that only THF contact the asphalt and that time in solution be kept to a minimum.

HP-CPC (Section C)

Columns (Section C,d) - We have used only columns by Waters, Inc., but other companies manufacture HP-GPC columns. The method should be reproducible on these columns, but we have no experience with them. It is likely that adjustments would have to be made in the composition of the column set and in flow rates. It is also likely that the chromatographic shapes will differ. However, the relationships between two asphalts should

be similar, at least in theory. (Some means of maintaining a stable temperature on the system is needed. This may be as sophisticated as a commercial thermostatted oven or as simple as a constant-temperature both of sufficient size to permit immersion of the solvent reservoir and from which fluid may be circulated around the column set, and through the refractometer.)

Detectors (Section C,f)

Our standard is based on the detection by ultraviolet light at 340 nm. Other detectors also give valid information, eg. refractive index. Chromatogram shape is different, of course, but can still be standardized. Refractive index detectors are very sensitive to temperature change and suffer from baseline drift, so it is necessary to thermostate this device.

If only a differential refractometer is used, the following changes must be made:

asphalt concentration 0.2% (w/v)

injection size 1 ml

RI attenuation - 8x

If UV and RI detectors are used in series, the concentration etc., suggested in the main procedure should be used. These are optimized for UV detection and a noisy signal should be expected from the RI detector.

Flow meter (Section C,g)

In lieu of a flow meter, internal standards may be used. A solution of 1% polystyrene (MW at least 35,000) and 0.06% toluene in THF may be used to dissolve the asphalt for analysis. This was the method we used in earlier projects and it has the advantage of showing flow rate changes visually on the chromatogram as changes in the elution time of the standards, especially relative to one another. It is important that the THF be the same as that being used in the HPLC. The standard solution should be freshly made no less frequently than every

other day and must be kept dry. Also, please note that the above concentrations were determined for use with the RI detector only and are likely to require some adjustment for use with the UV detector.

Pressure and/or flow rate changes may result from:

-an air bubble trapped in the system. Disconnect columns and increase flow rate. Be sure solvent is purged with helium.

-obstruction of frit in pre-column filter. Clean frit in THF in a sonicator. Use centrifuge to remove particulates from sample solutions.

-"dirty columns." Inject 2-3 ml pyridine (caution: toxic and odiferous) onto the columns and flush through with THF. This cleaning process should be routinely used after 30-40 samples.

Recording Data

Some means of recording the output of the detectors is required. This may be as simple as a strip-chart recorder, which provides data output which lends itself to visual comparisons but not so easily to computation of areas under the chromatographic curve.

Another alternative is a recording-integrator. This instrument will satisfy a desire for both visual and integrated area information.

A third alternative is a computer system which will perform the same functions as the recording integrator and will permit in addition the storage of data on disc for later manipulation.

Data Analysis

This is an analytical system which requires the use of a standard. For the purposes of analyzing asphalt cement, a standard asphalt is most convenient. An intra-laboratory standard should be selected with the following suggested criteria:

1) An aged asphalt extracted from a pavement.
2) If the asphalt is also to be a performance model, it should be representative of pavements which have given excellent performance in the climate Region of interest.
3) The supply of the asphalt should be stored in small containers with mouths wide enough to permit sampling by a small spatula. Repeated heating and excess exposure to air should be avoided.

The standard should be analyzed as the first sample of the day. (The results should be compared with previous analyses to monitor the condition of the system) and again as the last sample of the day, particularly if 5 or 6 samples have been analyzed.

Appendix C. Actual and Predicted Rut Depths and Values of Predictor Contributions